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SP : 6 (5) - 1980

HANDBOOK
FOR
STRUCTURAL ENGINEERS

5. COLD-FORMED, LIGHT-GAUGE STEEL STRUCTURES

(First Revision)

BUREAU OF INDIAN STANDARDS

HANDBOOK
FOR
STRUCTURAL ENGINEERS
NO. 5

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FOR

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5. COLD-FORMED, LIGHT-GAUGE STEEL STRUCTURES

(First Revision)

BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
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AMENDMENT NO. 1 MARCH 1984

TO

SP : 6 (5) -1980 HANDBOOK FOR STRUCTURAL ENGINEERS

5. COLD-FORMED, LIGHT-GAUGE STEEL STRUCTURES

(First Revision)

Corrigendum

(*Page* 39, *clause* **9.3.2**, *Equation* 34) — Substitute the following for the existing equation:

'For the non-linear portion, that is, between yield and elastic buckling, an allowable stress of $F_y = \frac{1\,275\sqrt{F_y}}{h/t}$ is permitted and the limit of h/t ratio is kept less than $\frac{4\,590}{\sqrt{F_y}}$' . . . (34)

(SMBDC : 7)

Printed at Simco Printing Press, Delhi, India

0. FOREWORD

0.1 This Handbook, which has been processed by the Structural Engineering Sectional Committee, SMBDC 7, the composition of which is given in Appendix A, has been approved for publication by Structural and Metals Division Council and Civil Engineering Division Council of ISI.

0.2 Steel, which is a very important basic raw material for industrialization, had been receiving attention from the Planning Commission even from the very early stages of the country's First Five Year Plan period. The Planning Commission not only envisaged an increase in production capacity in the country, but also considered the question of even greater importance, namely, the taking of urgent measures for the conservation of available resources. Its expert committees came to the conclusion that a good proportion of the steel consumed by the structural steel industry in India could be saved if more efficient procedures were adopted in the production and use of steel. The Planning Commission, therefore, recommended to the Government of India that the Indian Standards Institution should take up a Steel Economy Project and prepare a series of Indian Standardspecifications, handbooks, and codes of practices in the field of steel production and utilization.

0.3 Over several years of continuous study in India and abroad, and the deliberations at numerous sittings of committees, panels and study groups, have resulted in the formulation of a number of Indian Standards in the field of steel production, design and use, a list of which is given in Appendix B.

0.4 In comparison with conventional steel construction which utilizes standardized hot-rolled shapes, cold-formed, light-gauge steel structures are a relatively new development. To be sure, corrugated sheet, which is an example of such construction, has been used for many decades. However, systematic use had started in the United States only in the 1930's and reached large-scale proportions only after the Second World War. In Europe, such large-scale use is beginning only now in some countries.

0.5 The design of light-gauge structural members differs in many respects from that of other types of structures. Since its principles are relatively new, they are as yet not usually taught in engineering institutions. The important methods, referring to such design have been formulated in IS : 801-1975, to which reference has been made throughout.

0.6 Intelligent and economical use of a code by a designer may be made only if he has a thorough understanding of the physical behaviour of the structures to which the code applies, and of the basic information on which the code is based.

0.7 This handbook which deals with the use of cold-formed, light-gauge sections in structures was first published in 1970 and was based on the 1958 edition of IS : 801. With the revision of IS : 801 in 1975, a revision of the handbook was taken. This revision has been prepared in three sections:

Section 1	Commentary
Section 2	Design tables and design curves
Section 3	Design examples

0.7.1 Section 1 contains a systematic discussion of IS : 801-1975 and its background, arranged by fundamental topics in a manner useful to the practicing designer. This portion should enable the engineer not only to orient himself easily with the provisions of IS : 801-1975 but also to cope with design situations and problems not specifically covered in IS : 801-1975.

0.7.2 Section 2 contains considerable supplementary information on design practices in the form of tables and design curves based on provisions of IS : 801-1975.

0.7.3 Section 3 contains a number of illustrated design examples worked out on the basis of provisions of IS : 801-1975 and using various tables and design curves given in Section 2.

0.8 This handbook is based on, and requires reference to the following Indian Standards:

IS : 800-1962	Code of practice for use of structural steel in general building construction (*revised*)
IS : 801-1975	Code of practice for use of cold-formed light gauge steel structural members in general building construction (*first revision*)
IS : 811-1965	Specification for cold-formed light gauge structural steel sections (*revised*)
IS : 816-1969	Code of practice for use of metal arc welding for general construction in mild steel (*first revision*)
IS : 818-1968	Code of practice for safety and health requirements in electric and gas welding and cutting operations (*first revision*)

IS : 875-1964	Code of practice for structural safety of buildings: Loading standards (*revised*)
IS : 1079-1973	Specification for hot-rolled carbon steel sheet and strip (*third revision*)
IS : 1261-1959	Code of practice for seam welding in mild steel
IS : 4000-1967	Code of practice for assembly of structural joints using high tensile friction grip fasteners

0.9 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS : 2-1960*. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

0.10 In the preparation of this handbook, the technical committee has derived valuable assistance from commentary on the 1968 edition of the specification for the Design of Cold-Formed Steel Structural Members by George Winter published by American Iron and Steel Institute — New York.

---

*Rules for rounding off numerical values (*revised*).

1. SCOPE

1.1 This section contains a systematic discussion of the provisions of IS : 801-1975.

2. INTRODUCTION

2.1 Light gauge members are cold-formed from steel sheets or strips. Thickness for framing members (beams, joists, studs, etc) generally ranges from 1·2 to 4·0 mm; for floor and wall panels and for long span roof deck from 1·2 to 2·5 mm, and for standard roof deck and wall cladding from 0·8 to 1·2 mm. These limits correspond to normal design practice, but should not be understood to restrict the use of material of larger or smaller thickness. In India light gauge members are widely used in bus body construction, railway coaches, etc and the thickness of these members vary from 1·0 to 3·2 mm.

2.2 Forming is done in press brakes or by cold-rolling. Light gauge members can be either cold-formed in rolls or by press brakes from flat steel generally not thicker than 12·5 mm. For repetitive mass production they are formed most economically by cold-rolling, while small quantities of special shapes are most economically produced on press brakes. The latter process, with its great versatility of shape variation makes this type of construction as adaptable to special requirements as reinforced concrete is in its field use. Presently light gauge members are produced in India both by press brake system (for use in small quantities) and by cold-forming (for use in large quantities). These members are connected together mostly by spot welds, cold riveting and by special fasteners.

2.3 The cold-formed members are used in preference to the hot-rolled sections in the following situations:

a) Where moderate loads and spans make the thicker hot-rolled shapes uneconomical, for example, joists, purlins, girts, roof trusses, complete framing for one and two storey residential, commercial and industrial structures;

b) Where it is desired that load carrying members also provide useful surfaces, for example, floor panels and roof decks, mostly installed without any shoring and wall panels; and

c) Where sub-assemblies of such members can be prefabricated in the plant, reducing site erection to a minimum of simple operations, for example sub-assembly of panel framing up to 3 × 4 metres and more for structures listed in (a), standardized package shed type utility buildings, etc.

3. CURRENT SHAPES

3.1 In contrast to hot-rolling, the cold-forming processes coupled with automatic welding permit an almost infinite variety of shapes to be produced. The requirements for the sections generally manufactured in India are given in IS : 811-1965. But the freedom of designers is not limited to the use of sections listed in that standard. This is because a great variety of usages require a corresponding variety of shapes. However the designer is advised to seek the advice of the manufacturers or fabricators before specifying special sections.

3.2 Shapes for Structural Framing — Many of the shapes currently in use are shown in Fig. 1.

3.3 Shapes 1 to 21 in Fig. 1 are outlines similar to hot-rolled shapes, except that in shapes 2, 4 and 6 lips are used to stiffen the thin flanges. These shapes are easily produced but have the disadvantage of being unsymmetrical. Shapes 7 to 11 are to be found only in cold-formed construction, they have the advantage of being symmetrical. Shapes 7, 8, 10 and 11 are adapted for use in trusses and latticed girders; these sections are compact, well stiffened and have large radii of gyration in both principal directions. Shape 9, lacking edge stiffeners on the vertical sides is better adapted for use as a tension member. Shapes 12 and 13 are used specifically as girts and cave struts respectively, in all-metal buildings shape 12 being the same as shape 4 which is also used for purlins. The above members are all one-piece shapes produced merely by cold-forming.

3.3.1 When automatic welding is combined with cold rolling, it is possible to obtain additional shapes. Shapes 14 and 15 are two varieties of I shapes, the former better adapted for use as studs or columns, the latter for joists or beams. Two of the most successful shapes, namely shapes 16 and 17 are further adaptations of shapes 14 and 15. By deforming the webs and by using projection spot welding, curved slots are formed which provide nailing grooves for connecting collateral material, such as wall boards and wood floors. Shapes 18 and 19 represent closed members particularly favourable in compression the former primarily for columns, the latter for compression chords of trusses. Shape 20 shows one of a variety of open web joists, with chords shaped for nailing, and shape 21 shows sections similar to the chords of shape 20 connected directly to form a nailable stud.

3.3.2 The shapes in Fig. 1 do not exhaust the variety of sections now in use. There is no doubt that design ingenuity will produce additional shapes with better structural economy than many of those shown, or better adapted to specific uses. In the design of such structural sections the main aim is to develop shapes which combine economy of material (that is a favourable strength weight ratio) with versatility ease of mass production, and provision for effective and simple connection to other structural members or to non-structural collateral material or both of them.

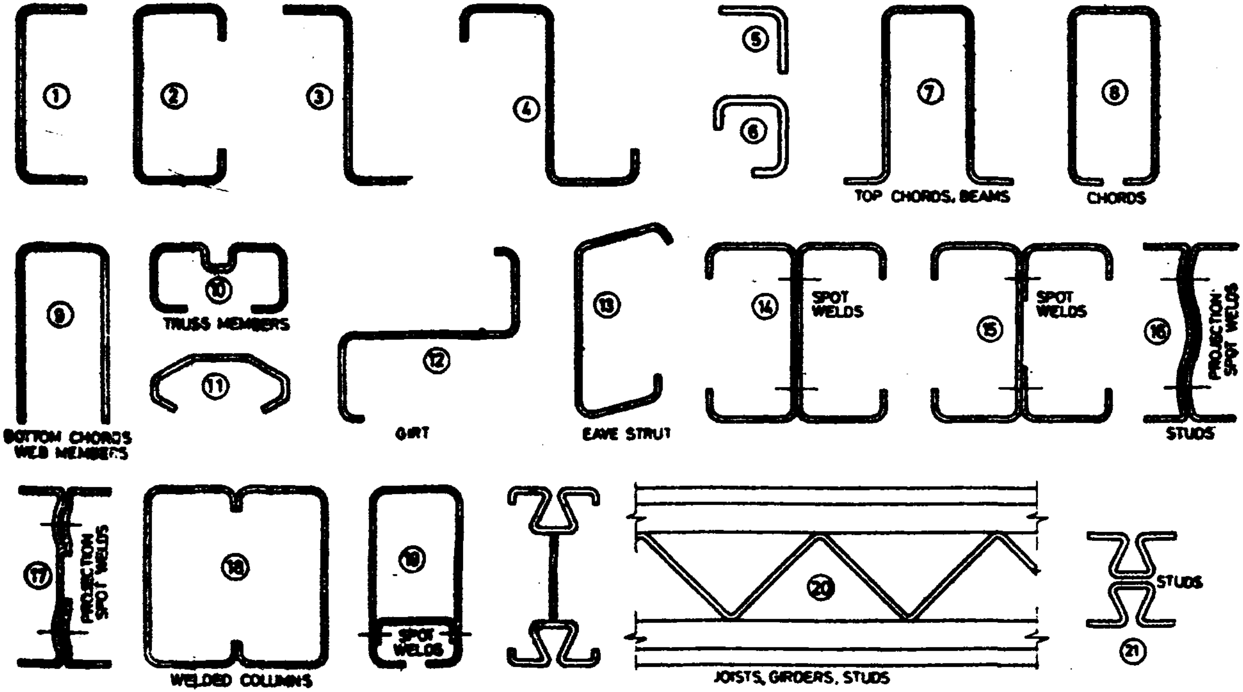
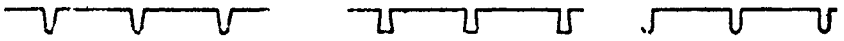


Fig. 1 Light-Gauge Shapes for Structural Framing

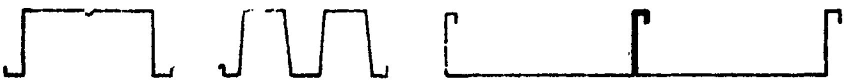
4. DECKS AND PANELS

4.1 Some typical roof decks, floor and roof panels, siding, and curtain wall panels as they have developed during the last 20 years in USA and are beginning to find application, duly modified, in other countries are shown in Fig. 2.

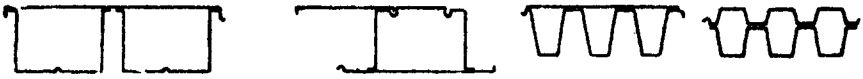
4.2 Standard roof decks are usually 58 mm deep, with a rib spacing of 130 mm and are used on spans between purlins up to 5 m. As compared to corrugated sheet they have the important advantage that the flat surface makes it possible to apply insulation and built-up roofing. Long-span roof decks are used for spans up to 6 m and more, which means that purlins in most cases may be dispensed with. Chief application is for industrial buildings, but also for other structures with relatively long roof spans, such as for schools.



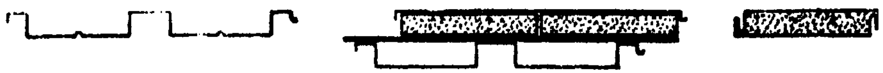
STANDARD ROOF DECKS



LONG - SPAN ROOF DECKS



FLOOR AND ROOF PANELS



CURTAIN - WALL PANELS

FIG. 2 FLOOR AND ROOF DECKS, AND WALL PANELS

4.3 Floor and roof panels are made to cover spans from 3 to 10 m. They are usually cellular in shape and permit a wide variety of ancillary uses. Thus, acoustic treatment is obtained by perforating bottom surfaces and installing sound absorbing elements, such as glass fibre insulation, in the cells. Electrification of the entire floor is achieved by permanent installation of wiring in the cells, which permits floor outlets to be placed wherever desired. Recessed lighting may be installed in the spaces between cells, etc. The flooring proper is installed on a light-weight concrete fill (50 to 75 mm) placed on top of the floor panels.

4.3.1 Curtain walls consist either of single-sheet siding or of cellular insulated wall panels.

4.4 Advantages of these systems are light weight which reduces the cost of main framing and foundations; speed of erection; absence of shoring or other temporary supports for floors and roofs; immediate availability; adaptability to later changes and additions; and suitability to perform enumerated ancillary functions.

4.5 In the design of these members, structural efficiency is only one of the many criteria since the shape should also be selected to minimize deflections, provide maximum coverage, permit adequate insulation, and accessibility of cells for housing conduits, etc. Optimum strength, that is, optimum strength-weight ratio, therefore, is desired only conditionally, that is, in so far as it is compatible with the other enumerated features.

4.6 It is evident from this discussion that the shapes used in lightgauge construction are quite different from, and considerably more varied than, those employed in hot-rolled framing. In consequence, an appropriate design code, such as IS : 801-1975 and IS : 800-1962 should enable the designer to compute properties and performance of practically any conceivable shape of cold-formed structural members.

5. MATERIAL

5.1 Structural steel sheet used for production of member should conform to IS : 1079 - 1973.

6. DEFINITIONS

6.1 **Stiffened Compression Element** — A flat compression element, for example, a plane compression flange of a flexural member (Fig. 3A, 3B and 3C) or a plane web or flange of a compression member, of which both edges parallel to the direction of stress are stiffened by a web, flange stiffening lip, intermediate stiffener or the like conforming to the requirement of **5.2.2** of IS : 801 - 1975.

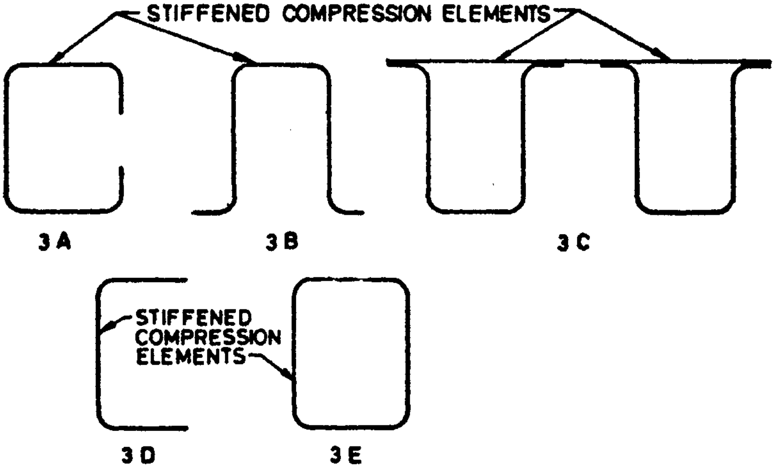

STIFFENED COMPRESSION ELEMENTS
3 A
3 B
3 C
STIFFENED COMPRESSION ELEMENTS
3 D
3 E

FIG. 3 STIFFENED COMPRESSION ELEMENTS

6.2 Unstiffened Compression Elements — A flat element which is stiffened at only one edge parallel to the direction of stress (Fig. 4).

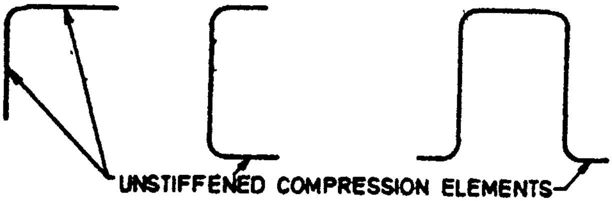


FIG. 4 UNSTIFFENED COMPRESSION ELEMENTS

6.3 Multiple Stiffened Elements and Subelements — An element that is stiffened between webs, or between a web and a stiffened edge (Fig. 5), by means of intermediate stiffeners which are parallel to the direction of stress and which conform to the requirements of 5.2.2 of IS : 801-1975. A subelement ist he portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.

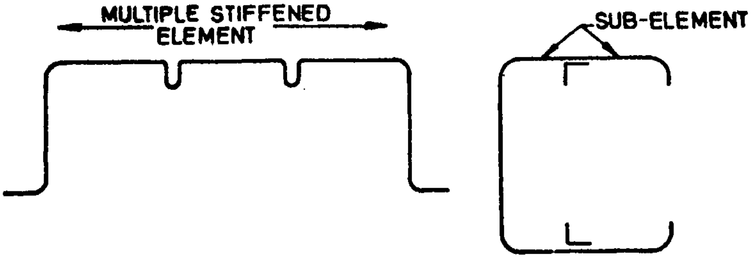
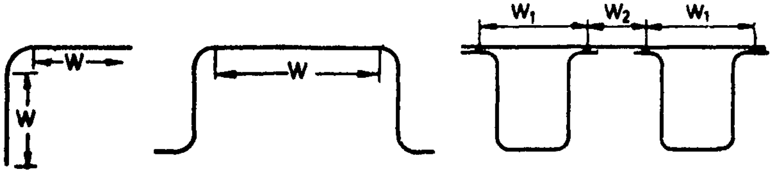


FIG. 5 MULTIPLE STIFFENED ELEMENT AND SUB-ELEMENT

6.4 Flat Width Ratio — The flat width ratio $\frac{w}{t}$ of a single flat element is the ratio of the flat width *w*, exclusive of edge fillets, to the thickness *t* (*see* Fig. 6).



$\frac{w}{t}$ $\frac{w}{t}$ $\frac{w_1}{t}$ or $\frac{w_2}{t}$

FIG. 6 FLAT WIDTH RATIO

6.5 Effective Design Width — Where the flat-width *w* of an element is reduced for design purposes, the reduced design width *b* is termed as the effective width or effective design width (Fig. 7).

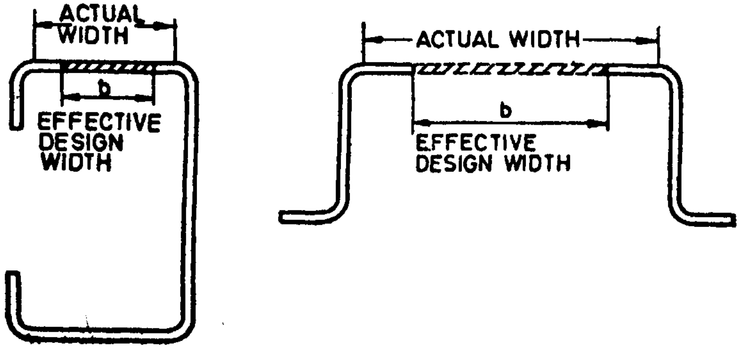
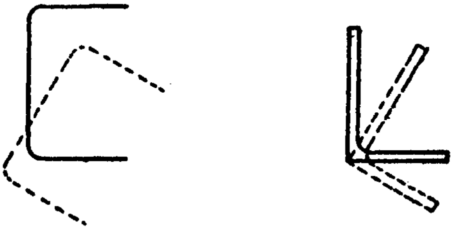


FIG. 7 EFFECTIVE DESIGN WIDTH

6.6 Torsional Flexural Buckling — A mode of buckling in which compression members can bend and twist simultaneously (Fig. 8).



(The cross section shown dotted after buckling)

Fig. 8 Torsional Flexural Buckling

6.7 Point Symmetric Section — A section symmetrical about a point (centroid), such as a 'Z' section having equal flanges (Fig. 9).

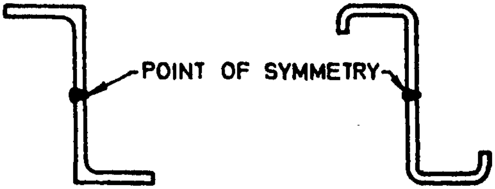


Fig. 9 Point Symmetric Section

6.8 Yield Stress, F_y — The cold-rolled steel sections are produced from strip steel conforming to IS : 1079-1973, the yield stresses of the steels are as follows:

Grade	*Yield Stress (Min)*
St 34	2 100 kgf/cm²
St 42	2 400 "
St 50	3 000 "
St 52	3 600 "

7. LOADS

7.1 For general guidance as to the loads to be taken into account in the design of structures, reference should be made to IS : 800-1962 and IS : 875-1964.

8. DESIGN PROCEDURE

8.1 General — All computations for safe load, stress deflection and the like shall be in accordance with conventional methods of structural design except as otherwise specified herein.

8.2 Properties of Sections — The properties of sections (cross-sectional area, moment of inertia, section modulus and radius of gyration) shall be determined in accordance with the conventional methods of structural design.

8.2.1 Computation of properties of formed sections may be simplified by using a method called 'linear method' in which the material of the section is considered concentrated along the central line of the steel sheet and the area elements replaced by straight or curved line elements. The thickness element t is introduced after the linear computation has been completed.

The total area of the section is found from the relation 'Area $= L_t \times t$' where L_t is the total length of all the elements. The moment of inertia of the section is found from the relation '$I = I' \times t$' where I' is the moment of inertia of the central line of steel sheet.

The section modulus is computed as usual by dividing I or ($I' \times t$) by the distance from neutral axis to the extreme and not to the central line of extreme element.

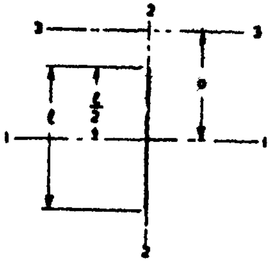
First power dimensions such as x, y and r (radius of gyration) are obtained directly by the linear method and do not involve the thickness dimension.

When the flat width w of a stiffened compression element is reduced for design purposes, the effective design width b is used directly to compute the total effective length $L_{effective}$ of the line elements.

The elements into which most sections may be divided for application of the linear method consist of straight lines and circular arcs. For convenient reference, the moments of inertia and location of centroid of such elements are identified in Fig. 10.

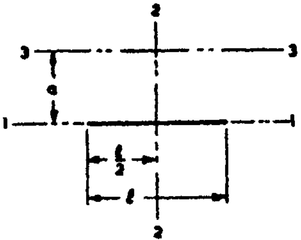
8.2.2 The formula for line elements are exact, since the line as such has no thickness dimensions; but in computing the properties of an actual section, where the line element represents an actual element with a thickness dimension, the results will be approximate for the following reasons:

a) The moment of inertia of a straight actual element about its longitudinal axis is considered negligible.



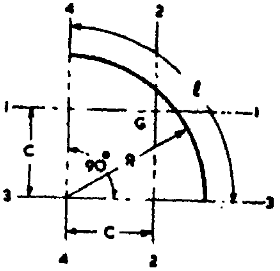
$I_1 = \frac{l^3}{12}$ $\quad I_2 = 0$

$I_3 = la^2 + \frac{l^3}{12} = l\left(a^2 + \frac{l^2}{12}\right)$



$I_1 = 0$ $\quad I_2 = \frac{l^3}{12}$

$I_3 = la^2$

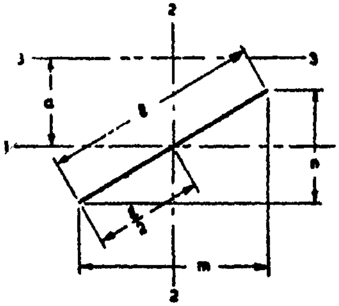


$l = 1{\cdot}57\,R$; $\quad C = 0{\cdot}637\,R$

$I_1 = I_2 = 0{\cdot}149\,R^3$

$I_3 = I_4 = 0{\cdot}785\,R^3$

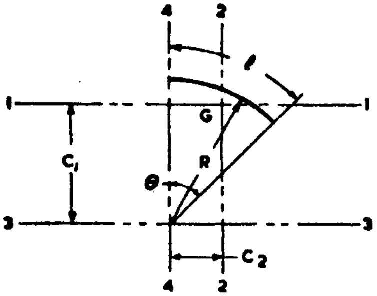
G = centre of gravity



$I_1 = \frac{ln^2}{12}$; $\quad I_2 = \frac{lm^2}{12}$

$I_3 = la^2 + \frac{ln^2}{12} = l\left(a^2 + \frac{n^2}{12}\right)$

FIG. 10 PROPERTIES OF LINE ELEMENTS — *Continued*



θ (expressed in radians) = 0·017 45 × θ (expressed in degrees and decimals thereof)

$$l = \theta R$$

$$C_1 = \frac{R \sin \theta}{\theta}$$

$$C_2 = \frac{R (1 - \cos \theta)}{\theta}$$

$$I_1 = \left[\frac{\theta + (\sin \theta) (\cos \theta)}{2} - \frac{(\sin \theta)^2}{\theta} \right] R^3$$

$$I_2 = \left[\frac{\theta - (\sin \theta) (\cos \theta)}{2} - \frac{(1 - \cos \theta)^2}{\theta} \right] R^3$$

$$I_3 = \left[\frac{\theta + (\sin \theta) (\cos \theta)}{2} \right] R^3$$

$$I_4 = \left[\frac{\theta - (\sin \theta) (\cos \theta)}{2} \right] R^3$$

G = centre of gravity

FIG. 10 PROPERTIES OF LINE ELEMENTS

b) The moment of inertia of a straight (actual) element inclined to the axis of reference is slightly larger than that of the corresponding line element, but for elements of similar length the error involved is even less than the error involved in neglecting the moment of inertia of the element about its longitudinal axis. Obviously, the error disappears when the element is normal to the axis.

c) Small errors are involved in using the properties of a linear arc to find those of an actual corner, but with the usual small corner radii the error in the location of the centroid of the corner is of little importance, and the moment of inertia generally negligible. When the mean radius of a circular element is over four times its thickness, as for tubular sections and for sheets with circular corrugations, the error in using linear arc properties practically disappears.

A typical worked out example is given in Section 3.

8.3 Effective Design Width — Consider a plate simply supported on two edges and loaded as shown in Fig. 11.

As the load q is gradually increased, the stress will be uniform. At a stress equal to the critical stress namely $f_{cr} = \dfrac{\pi^2 E}{3(1-\mu^2)(b/t)^2}$ ... (1) (where μ = Poisson's ratio and t = thickness) the plate at the centre will buckle. The stress distribution is as shown in Fig. 12A.

As the load q is gradually increased the unbuckled portion of the plate resists the loads and the distribution of stress is as shown in Fig. 12B. Failure occurs at a stage when the stress at the supported edge reaches yield stress F_y and the distribution of stress at this stage is as shown in Fig. 12C.

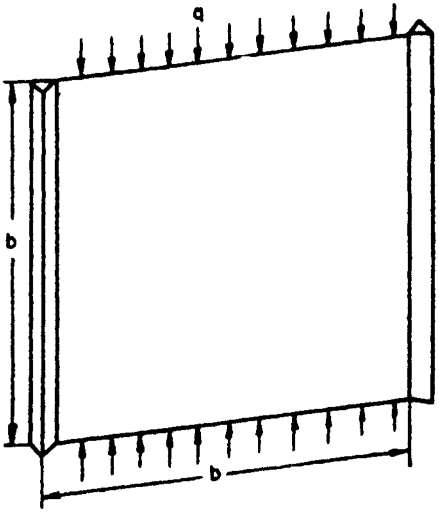


FIG. 11

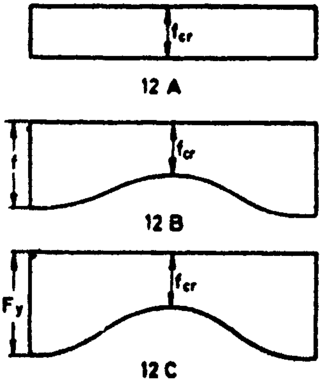
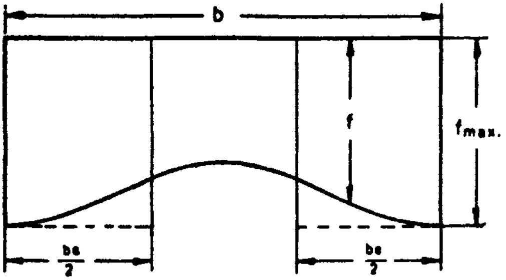


FIG. 12

For design purposes the total force is assumed to be distributed over lesser width with uniform stress. This reduced width is called the effective design width of plate (*see* Fig. 13).



$$\int_0^b f\,dx = f_{\max} \cdot b_e$$

FIG. 13

The simplest form for effective width expression is obtained by equating yield stress

$$F_y = \frac{\pi^2 E}{3\,(1-\mu^2)\left(\frac{b}{t}\right)^2}\text{, or}$$

$$\left(\frac{b}{t}\right)^2 = \frac{\pi^2 E}{3\,(1-\mu^2)\,F_y}\text{, or}$$

$$\left(\frac{b}{t}\right) = 1{\cdot}9\sqrt{\frac{E}{F_y}} \qquad \ldots\ldots (2)$$

This expression, known as Von Karman equation, based on experiments has been modified by Winter as

$$b = 1{\cdot}9t\sqrt{\frac{E}{f_{\text{Max}}}}\left[1 - 0{\cdot}475\left(\frac{t}{w}\right)\sqrt{\frac{E}{f_{\text{Max}}}}\right] \qquad \ldots\ldots (3)$$

IS : 801-1975 is based on the latest expression adopted by AISI Code 68 which is given as $\frac{b}{t} = 1{\cdot}9\sqrt{\frac{E}{f_{\text{Max}}}}\left[1 - 0{\cdot}415\left(\frac{t}{w}\right)\sqrt{\frac{E}{f_{\text{Max}}}}\right]$ ...... (4)

Substituting for E as 2 074 000 kgf/cm^2

$$\frac{b}{t} = \frac{2\,736}{\sqrt{f}}\left(1 - \frac{598}{(w/t)\sqrt{f}}\right) \qquad \ldots\ldots (5)$$

8.3.1 Formulae for load and deflection determination:

The stiffened compression elements fail when the edge stress (that is, the stress on the effective area) reaches the yield point. In order to compute the failure moment M_{ult} of a beam it is necessary to calculate the section modulus at a stress equal to the failure stress, that is the yield stress, and multiply it by the yield stress.

$$M_{ult} = S_{at\,F_y} \times f_y \qquad \ldots\ldots (6)$$

The factor of safety for bending members = 1·67 that is $F_y = 1{\cdot}67 \times f_b$ where f_b if the basic design stress.

$$M_{ult} = S_{at\,F_y} \times 1{\cdot}67 F_b \qquad \ldots\ldots (7)$$

$$\therefore\ M_{allowable} = \frac{M_{ult}}{\text{Factor of safety}}$$

$$= \frac{M_{ult}}{1{\cdot}67} = S_{at\,F_y} \times f_b \qquad \ldots\ldots (8)$$

It may be confusing to the designer to calculate allowable bending moment at section modulus at F_y and then multiply by f_b. To avoid this confusion, the effective width expression for load determination is modified by replacing f by 1·67 f so that the designer can substitute f for determining the effective width and thus calculate the section modulus and multiply the section modulus by f again.

Therefore, the expression for effective width for load calculation is obtained as follows by substituting 1·67f instead of f in expression (5):

$$\frac{b}{t} = \frac{2\,736}{\sqrt{1{\cdot}67 f}}\left[1 - \frac{598}{\frac{w}{t}\sqrt{1{\cdot}67 f}}\right]$$

$$= \frac{2\,117}{\sqrt{f}}\left[1 - \frac{462}{\frac{w}{t}\sqrt{f}}\right] \qquad \ldots\ldots (9)$$

The expressions (9) and (5) are rounded off and modified to arrive at the expression given in **5.2.1.1** of IS : 801-1975.

Load determination:

$$\frac{b}{t} = \frac{2\,120}{\sqrt{f}}\left[1 - \frac{465}{\frac{w}{t}\sqrt{f}}\right] \qquad \ldots\ldots (10)$$

Deflection determination:

$$\frac{b}{t} = \frac{2\,710}{\sqrt{f}}\left[1 - \frac{600}{\frac{w}{t}\sqrt{f}}\right] \qquad \text{......(11)}$$

The flanges are fully effective when $b = w$; substituting in the expression (10) as $b = w$;

$$\frac{w}{t} = \frac{2\,120}{\sqrt{f}}\left[1 - \frac{465}{\frac{w}{t}\sqrt{f}}\right] \qquad \text{......(12)}$$

By simplifying the expression (12)

$$\left(\frac{w}{t}\right)^2 - \frac{2\,120}{\sqrt{f}}\left(\frac{w}{t}\right) + \frac{985\,800}{f} = 0$$

This is a quadratic equation in $\left(\frac{w}{t}\right)$ and solving

$$\frac{w}{t} = \frac{1\,431}{\sqrt{f}} \qquad \text{......(13)}$$

This is modified as $\left(\frac{w}{t}\right)_{\text{lim}} = \frac{1\,435}{\sqrt{f}}$

In a similar way $\left(\frac{w}{t}\right)_{\text{lim}}$ for deflection determination can be obtained from expression (11) as,

$\left(\frac{w}{t}\right)_{\text{lim}} = \frac{1\,813}{\sqrt{f}}$ which is modified as

$\left(\frac{w}{t}\right)_{\text{lim}} = \frac{1\,850}{\sqrt{f}}$ in IS : 801-1975

8.3.2 *Effective Widths for Square and Tubular Sections* — These sections being rolled under strict quality control, a higher value of effective widths are permitted to be in agreement with the experimental results.

8.3.3 *Multiple Stiffened Elements and Wide Stiffened Elements with Edge Stiffeners* — The elements with large flat width ratios become uneconomical because they have only very small effective widths. In such cases the elements may be stiffened with stiffeners as shown in Fig. 5. In cases when flat width ratio of subelement exceeds 60 because of the shear lag effect, the effective design width and also the effective area of the stiffener should be reduced as given in **5.2.1.2** of IS : 801-1975.

8.3.4 *Stiffeners for Compression Elements* — In order that a flat compression element may be considered a stiffened compression element it shall be stiffened along the edge with stiffener of sufficient rigidity. The minimum moment of inertia required to stiffen the edge has been calculated approximately and the expression under **5.2.2.1** of IS : 801-1975 has been arrived at. The experimental results give a close fit to the values obtained from the expression. Whereas an edge stiffener stiffens only one compression element, an intermediate stiffener stiffens the two compression elements on either side of the stiffener. The minimum moment of inertia required for an intermediate stiffener is proposed as double the moment of inertia of an edge stiffener.

Tests have shown that in a member with intermediate stiffeners the effective width of a subelement is less than that of an ordinary stiffened element of the same $\frac{w}{t}$ ratio, particularly if $\frac{w}{t}$ exceeds about 60. This may be understood from the discussion in the following paragraphs.

In any flanged beam the normal stresses in the flanges are the result of shear stresses between web and flange. The web, as it were, originates the normal stresses by means of the shear it transfers to the flange. The more remote portions of the flange obtain their normal stress through shear from those closer to web, and so on. In this sense there is a difference between webs and intermediate stiffeners in that the latter is not a shear-resisting element and therefore does not 'originate' normal stresses through shear. On the contrary, any normal stress in the stiffener should have been transferred to it from the web or webs through the intervening flange portions. As long as the subelement between web and stiffener is flat or only very slightly buckled $\left(\text{that is with low } \frac{w}{t}\right)$ this shear proceeds unhampered. In this case, then, the stress at the stiffener is equal to that at the web and the subelement is as effective as a regular stiffened element of the same $\frac{w}{t}$ ratio.

However for large $\frac{w}{t}$ ratios the slight buckling waves of the subelement interfere with complete shear transfer and create a shear lag, consequently the stress distribution in a multiple stiffened element, when the $\frac{w}{t}$ ratios of the subelements exceed about 60, can be thought of as represented in Fig. 14. That is, since the edge stress of a subelement is less at the stiffener than at the edge, its effective width is less than that of corresponding stiffened element $\left(\text{with same } \frac{w}{t} \text{ ratio}\right)$. Also the efficiency of the stiffener itself is reduced by this lower stress; this fact is best accounted for by assigning a reduced effective area to the stiffener.

Correspondingly the effective widths of subelements are identical with those obtained from **5.2.1.1** of IS : 801-1975 only where $\frac{w}{t}$ is less than 60. For larger $\frac{w}{t}$ ratios these effective widths are reduced according to the formula **5.2.1.2** of IS : 801-1975. Also in view of the reduced efficiency of the intermediate stiffeners as just described, their effective area for determining properties of sections of which they are part, is to be determined from the formula for A_{eff}. It should be noted that the usually slight reduction in efficiency provided by **5.2.1.2** of IS : 801-1975 does not detract from the very considerable gain structural economy obtained by intermediate stiffeners.

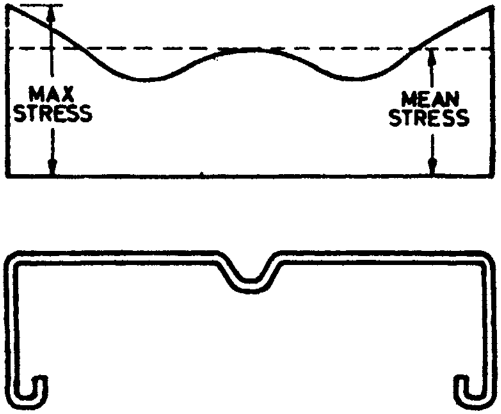


FIG. 14 MULTIPLE STIFFENED ELEMENT

Provisions (a), (b) and (c) of **5.2.2.2** of IS : 801-1975 reflect the described situation, namely, that the intermediate stiffeners, due to shear lag across slightly waved subelement are not as effective as complete webs would be. Consequently, if a number of stiffeners were placed between webs at such distances that the resulting subelements have $\frac{w}{t}$ ratios of considerable magnitude, there would be a rapidly cumulative loss of effectiveness with increasing distance from the web. Provisions (a) and (b) in essence provide that if $\frac{w}{t}$ of the subelements exceeds $\left(\frac{w}{t}\right)_{\text{lim}}$, that is, if they are in the slightly buckled state so that the shear transfer is interfered with, only such intermediate stiffeners which are adjacent to web shall be regarded as effective. On the other hand if stiffeners are so closely spaced that the subelements show no tendency to slight buckle $\left[\text{that is, } \frac{w}{t} \text{ is less than } \left(\frac{w}{t}\right)_{\text{lim}}\right]$, the entire element including stiffeners will be fully effective. This is what provision (c) also specifies for such closely stiffened elements an effective thickness t_s for computing, when needed, the flat width ratio of entire element (including stiffeners). It is easily checked that this t_s is the thickness of a solid plate having the same moment of inertia as the actual, closely stiffened element.

9. ALLOWABLE DESIGN STRESSES

9.1 Compression on Unstiffened Elements — An unstiffened compression element may fail in yielding if it is short and its $\frac{w}{t}$ ratio is less than a certain value.

The elastic critical local buckling stress for a uniformly compressed plate is

$$f_{cr} = \frac{K\pi^2 E}{12\,(1-\mu^2)\left(\frac{w}{t}\right)^2} \qquad \text{...... (14)}$$

For a long rectangular plate with a free edge and supported on three edges the value of $K = 0.425$. When the restraining effect of the connected edge is considered K can be taken as 0·5. The limit of $\frac{w}{t}$ ratio below which the steel will yield can be found out by equating

$$F_y = \frac{0.5\pi^2 E}{12\,(1-\mu^2)\left(\frac{w}{t}\right)^2} \qquad \text{...... (15)}$$

Substituting for E and μ, $\left(\frac{w}{t}\right) = \frac{968.1}{\sqrt{F_y}}$ ...... (16)

as $E = 2\,074\,000$ kgf/cm^2 and $\mu = 0.3$.

If the steel has sharp yielding and the element is ideally plane, the element will fail by yielding below this limit. In practice the element will buckle below this theoretical limit and it has been found at a value of about 0·55 times this value will be suitable for practical cases and hence the limit is fixed as $\frac{530}{\sqrt{F_y}}$ in 6.2 of IS : 801-1975. As the cold-forming process sets up residual stresses this also reduces the proportional limit. By assuming a proportional limit of 0·65 F_y the limit of $\left(\frac{w}{t}\right)$ at which elastic buckling starts can be found out as

$$0{\cdot}65\,F_y = \frac{0{\cdot}5\pi^2 E}{12\,(1-\mu^2)\left(\frac{w}{t}\right)_e^2}$$

$$\left(\frac{w}{t}\right)_e = \sqrt{\frac{0{\cdot}5\pi^2 E}{7{\cdot}8\,(1-\mu^2)\,F_y}} \qquad \text{...... (17)}$$

Substituting $E = 2\,074\,000$ kgf/cm² and $\mu = 0\ 3$,

$$\left(\frac{w}{t}\right)_e = \frac{1\,200}{\sqrt{F_y}}$$

This limit is taken as $\frac{1\,210}{\sqrt{F_y}}$ in 6.2(b) of IS : 801-1975. For the stresses within the limit of $\frac{w}{t} = \frac{530}{\sqrt{F_y}}$ to $\frac{w}{t} = \frac{1\,210}{\sqrt{F_y}}$ that is the region of inelastic buckling line B in Fig. 15. Straight line variation is assumed and the equation is worked out as follows:

Let the equation to straight line be

$$f = m\left(\frac{w}{t}\right) + c$$

$$\text{at } \frac{w}{t} = \frac{530}{\sqrt{F_y}} \qquad F_a = \frac{F_y}{1{\cdot}67} = 0{\cdot}6\,F_y$$

$$\text{at } \frac{w}{t} = \frac{1210}{\sqrt{F_y}} \qquad F_a = \frac{0{\cdot}5\pi^2 E}{12\,(1-\mu^2)\left(\frac{1\,210}{\sqrt{F_y}}\right)^2 \times 1{\cdot}67} = 0{\cdot}383\,F$$

$$\therefore 0{\cdot}6\,F_y = m\frac{530}{\sqrt{F_y}} + c$$

$$\text{and } 0{\cdot}383\,F_y = m\frac{1210}{\sqrt{F_y}} + c$$

Solving these equations:

$m = -0{\cdot}000\,32\,F_y\sqrt{F_y}$ and

$c = 0{\cdot}769\,F_y$

Hence substituting in the equation, the expression for allowable stress is obtained as:

$$F_a = F_y\left[0{\cdot}769 - \left(\frac{3{\cdot}20}{10^4}\right)\frac{w}{t}\sqrt{F_y}\right]$$

This is rounded off as the expression given in IS : 801-1975

$$F_a = F_y\left[0{\cdot}767 - \left(\frac{3{\cdot}15}{10^4}\right)\frac{w}{t}\sqrt{F_y}\right] \qquad \text{..... (18)}$$

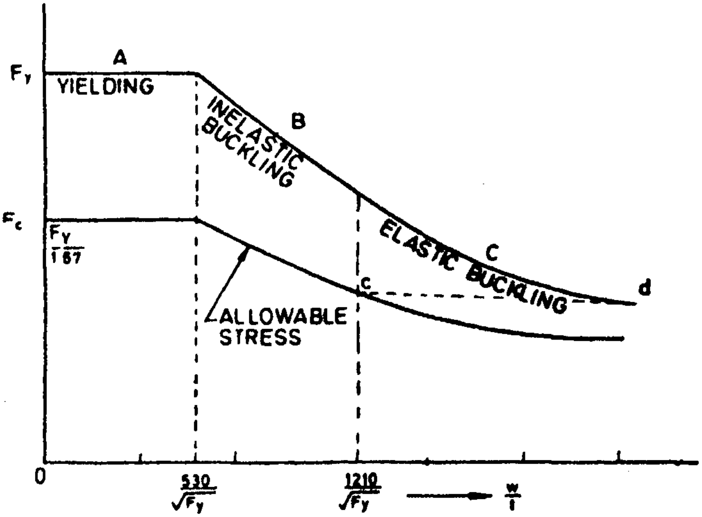


Fig. 15 Unstiffened Element Failure Stresses and Allowable Stresses for $0 < \frac{w}{t} < 60$

For $\frac{w}{t}$ ratio from 25 to 60 the allowable stress is obtained as dividing the expression $\frac{0{\cdot}5\pi^2 E}{12\,(1-\mu^2)\,(w/t)^2}$ by a factor of safety of 1·67

$$\text{that is, } F_a = \frac{0{\cdot}5\pi^2 E}{12\,(\,1-\mu^2\,)\,(\,w/t\,)^2 \times 1{\cdot}67}$$

$$= \frac{561\,250}{\left(\frac{w}{t}\right)^2} \qquad \ldots\ldots(19)$$

The expression given in **6.2** (d) of IS : 801-1975 is $\frac{562\,000}{\left(\frac{w}{t}\right)^2}$.

For sections other than angle sections the allowable stress expression is obtained by joining the point *c* and *d*. As for large $\frac{w}{t}$ ratios there is sufficient post buckling strength factor of safety which is taken care of in the post buckling strength and the point *d* is taken in the buckling curve.

By fitting a straight line between the limits $w/t = 25$ and 60, the equation to straight line is obtained as $\left(1\,390 - 20\,\frac{w}{t}\right)$ ...... (20)

9.2 Laterally Unbraced Beams — The critical moment for a beam simply supported at the two ends and subjected to two end couples is

$$M_{cr} = \frac{\pi}{L}\sqrt{EI_y\,GJ\left(1+\frac{\pi^2 EC_w}{GJL^2}\right)} \qquad \ldots\ldots(21)$$

$$\text{For I beams } C_w = \frac{b^3\,td^2}{24}$$

$$\text{and } I_y = \frac{b^3 t}{6}$$

$$\text{The equation (21) becomes } M_{cr} = \frac{\pi}{L}\sqrt{EI_y\,GJ + \frac{\pi^2 E^2}{L^2}\,I_y\,\frac{b^3 td^2}{24}}$$

$$= \frac{\pi}{L}\sqrt{EI_yGJ + \left(\frac{\pi}{L}\right)^2 I_y^2\cdot E^2\,\frac{d^2}{4}} \qquad \ldots\ldots(22)$$

Therefore, the critical stress for lateral buckling of an I beam subjected to pure bending is given by

$$\sigma_{cr} = \frac{M_{cr}}{S_x} = \frac{M_{cr}}{\left(\frac{I_x}{d/2}\right)} = \frac{\pi d}{2\,I_x L}\sqrt{EI_yGJ + \frac{\pi^2\,E^2}{L^2}\,I_y^2\frac{d^2}{4}}$$

$$= \frac{\pi^2 d^2 E^2}{2L^2}\sqrt{\frac{I_yGJ}{I_x^2 Ed^2}\cdot\frac{L^2}{\pi^2} + \frac{I_y^2}{4I_x^2}} \qquad \ldots\ldots(23)$$

For thin-walled sections the first term appearing in the square root is considerably less than the second term and hence neglecting the first term, we get

$$\sigma_{cr} = \frac{\pi^2 d^2 E}{2L^2} \quad \frac{I_y}{2\,I_x}$$

$$= \frac{\pi^2 E d^2}{2\,L^2} \quad \frac{I_y}{2\left(S_x \frac{d}{2}\right)}$$

$$= \frac{\pi^2 E d}{S_x\,L^2} \cdot \frac{I_y}{2} = \frac{\pi^2 E d}{S_{xo}\,L^2}\,I_{yo} \qquad \ldots .\ (24)$$

Where S_{xo} is the section modulus with respect to the compression flange and I_{yo} is the moment of inertia of compression flange about YY-axis that is

$$I_{yo} = \frac{I_y}{2}$$

To consider the effect of other end conditions coefficient C_b is added and

$$\sigma_{cr} = \frac{\pi^2 E\,C_b}{\dfrac{S_{xo}\,L^2}{d\,I_{yo}}} \qquad \ldots\ldots\ (25)$$

It may be noted that the equation applies to the elastic buckling of cold-formed steel beams when the computed theoretical buckling stress is less than or equal to the proportional limit σ_{pr}. But if the computed stress exceeds the proportional limit then the beam will fail by inelastic buckling. For extremely short beams the maximum moment capacity may reach full plastic moment M_p. A study by Galambos* has shown that for wide flanged beams $M_p = 1{\cdot}11\,M_y$.

This means that extreme fibre stress may reach an equivalent value of $1{\cdot}11\,f_y$ when $\frac{L^2\,S_{xo}}{d\,I_{yo}} = 0$, if we use the elastic section modulus S_{xo}.

As in the case of compression members, effective proportional limit can be assumed as one half the maximum stress that is

$$\sigma_{pr} = \tfrac{1}{2}\,(\,1{\cdot}11\,F_y\,) = 0{\cdot}555\,F_y \qquad \ldots\ (26)$$

The value of corresponding $\frac{L^2\,S_{xo}}{d\,I_{yo}}$ is obtained as

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*Inelastic lateral buckling of beams. T. V. Galambos. Journal of Structural Division ASCE Proc. Volume 89, No. ST 5 October 1963.

$$\frac{\pi^2 E C_b}{\dfrac{L^2 S_{xc}}{d I_{yc}}} = 0{\cdot}555 F_y$$

or $$\frac{L^2 S_{xc}}{d I_{yc}} = \frac{\pi^2 E C_b}{0{\cdot}555 F_y} = \frac{1{\cdot}8 \pi^2 E C_b}{F_y} \qquad \ldots\ldots (27)$$

The stress against $\sqrt{\dfrac{L^2 S_{xc}}{d I_{yc}}}$ is shown in Fig. 16.

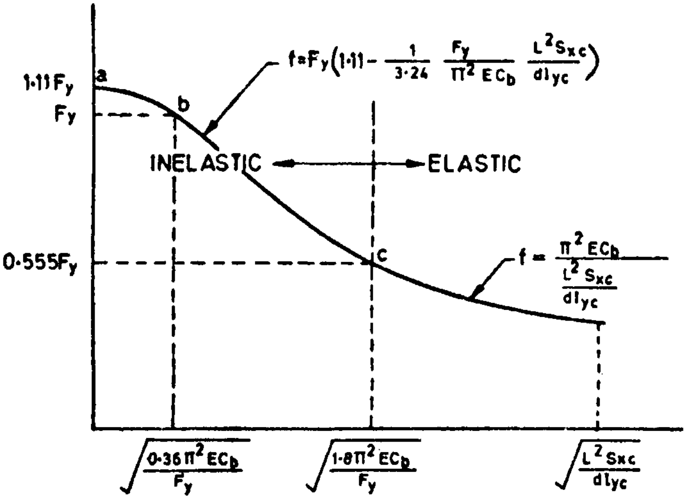


FIG. 16

The equation for the stress in the inelastic region is obtained by fitting a parabola $f = F_y \left[A - \dfrac{1}{B} \cdot \dfrac{F_y}{f_{cr}} \right]$ between the points *a* and *c*

At point *a*, $\dfrac{L^2 S_{xc}}{d I_{yc}} = 0$ and $f = 1{\cdot}11 F_y$ and

at point *c*, $\dfrac{L^2 S_{xc}}{d I_{yc}} = \dfrac{1{\cdot}8 \pi^2 E C_b}{F_y}$ and $f = 0{\cdot}555 F_y$. Substituting these, the value of *A* is found as 1·11 and that of *B* as 3·24.

The value of $\sqrt{\dfrac{L^2 S_{xo}}{d\,I_{yo}}}$ at which the stress $f = F_y$ is found out as

$$F_y = F_y \left[\, 1{\cdot}11 - \frac{1}{3{\cdot}24} \cdot \frac{F_y}{\pi^2 E\, C_b} \cdot \frac{L^2 S_{xo}}{d\, I_{yo}} \right],$$

$$\text{solving } \frac{L^2 S_{xo}}{d I_{yo}} = 0{\cdot}36 \frac{\pi^2 E\, C_b}{F_y}$$

For the allowable stresses in **6.3** of IS : 801-1975, the stresses are obtained by dividing the following expression by the factor of safety 1·67. For the inelastic range the expression is

$$f = F_y \left[\, 1{\cdot}0 - \frac{1}{3{\cdot}24}\ \frac{F_y}{\pi^2 E\, C_b}\ \frac{L^2 S_{xo}}{d\, I_{yo}} \right] \qquad \ldots\ldots(28)$$

In this expression to be on safe side the factor 1 is taken instead of 1·11 and the expression in the elastic range is $f = \dfrac{\pi^2 E\, C_b}{\dfrac{L^2 S_{xo}}{d\, I_{yo}}}$ ......(29)

By dividing the expressions in (28) by 1·67,

$$F_b = \tfrac{2}{3} F_y - \frac{F_y^2}{5{\cdot}4\pi^2 E C_b} \left(\frac{L^2 S_{xo}}{d\, I_{yo}} \right)$$

for the range $\dfrac{L^2 S_{xo}}{d\, I_{yo}}$ greater than $\dfrac{0{\cdot}36\pi^2 E\, C_b}{F_y}$ and $> \dfrac{1{\cdot}8\pi^2 E\, C_b}{F_y}$. When $\dfrac{L^2 S_{xo}}{d\, I_{yo}} > \dfrac{1{\cdot}8\pi^2 E\, C_b}{F_y}$ that is elastic range

$$F_b = 0{\cdot}6\ \frac{\pi^2 E\, C_b}{\dfrac{L^2 S_{xo}}{d\, I_{yo}}} \qquad \ldots\ldots(30)$$

For $\dfrac{L^2 S_{xo}}{d\, I_{yo}} < \dfrac{0{\cdot}36\ \pi^2 E\, C_b}{F_y}$ allowable stress is naturally $\dfrac{F_y}{1{\cdot}67} = f_b$

9.2.1 The Z-shaped sections, when they are loaded parallel to web they deflect laterally due to unsymmetrical bending, if not properly braced.

Hence to be on conservative side the value as given in **6.3**(b) of IS : 801-1975 are assumed.

9.3 Webs of Beams

9.3.1 In regard to webs, the designer is faced with somewhat different problems in light gauge steel construction than in heavy hot-rolled construction. In the latter, the webs with large *h*/*t* ratios are usually furnished with stiffeners to avoid reduction of allowable stress. In contrast in cold-formed construction large h/t ratios are the rule rather than the exception. At the same time the fabrication process, as a rule, makes it difficult, though not impossible, to employ stiffeners. Under these conditions the problem is that of so limiting the various allowable web stresses that adequate stability is obtained without the use of stiffeners.

9.3.2 The web of a beam may be considered as a simply supported plate subjected to shear only. The elastic stress at which a simply supported plate subjected to shear is $\sigma_{cr} = \frac{5\cdot35\,\pi^2 E}{12\,(1-\mu^2)\,(h/t)^2}$ where h is the smaller dimension and t, the thickness.

Substituting for the value of E = 2 074 000 kgf/cm² and $\mu = 0\cdot3$;

$$\sigma_{cr} = \frac{5\cdot35 \times 9\cdot87 \times 2\,074\,000}{12 \times 0\cdot91\,(h/t)^2} = \frac{10\,028\,980}{(h/t)^2}$$

Assuming a factor of safety equal to 1·71. Allowable stress in elastic shear buckling is:

$$\frac{10\,028\,980}{1\cdot71\,(h/t)^2} = \frac{5\,864\,904}{(h/t)^2} \qquad \ldots\ldots(31)$$

This value is given in **6.4.1**(b) of IS : 801-1975 as $F_v = \frac{5\,850\,000}{(h/t)^2}$ ......(32)

The yielding stress in shear is known to be $\frac{1}{\sqrt{3}}$ times that of yielding stress in tension $F_{ys} = \frac{F_y}{\sqrt{3}}$.

In the yielding case, that is for smaller h/t ratios, a lesser factor of safety is permitted and is equal to 1·44.

Hence for smaller h/t ratios, that is, when the sheet fails by yielding by shear, the allowable stress $= \frac{F_y}{\sqrt{3} \times 1\cdot44} = 0\cdot4\,F_y$ ......(33)

This is given as the maximum limit in **6.4.1**(a) of IS : 801-1975.

For the non-linear portion, that is, between yield and elastic buckling, an allowable stress of $F_v = \frac{1\,275}{h/t}$ is permitted and the limit of h/t ratio is kept less than $\frac{4\,590}{\sqrt{F_y}}$ ......(34)

9.4 Compression Members

9.4.1 *General* — The basic difference between a compression member in hot-rolled section and cold-formed section is that, in cold-formed light gauge sections, as the width-thickness ratios of component elements of cross section are large, these elements will be undergoing local buckling also. Hence it is necessary to incorporate the local buckling effects in the allowable stress expressions. This is done by incorporating a factor Q in the allowable stress expressions.

9.4.2 *Axial Stress in Compression* — In light gauge sections because of the possibility of local buckling a factor Q which is less than 1 is associated with the yield stress F_y and if we substitute QF_y for F_y in the well known axial compressive expression, the expressions given in 6.6 of IS : 801-1975 can be obtained.

9.4.2.1 *To find the value of* Q

a) *For stiffened elements* — For members composed entirely of stiffened elements:

$$P_{ult} = A_{eff} \times F_y$$

where

P_{ult} is the yield load

$$\frac{P_{ult}}{A} = \frac{A_{eff}}{A} . F_y \qquad \text{...... (35)}$$

Comparing with the expression yield stress $= Q.F_y$

$\therefore Q = \frac{A_{eff}}{A}$ for stiffened element where A_{eff} is the effective area of all stiffened elements computed for basic design stress.

b) *For unstiffened elements* — When the member consists of unstiffened elements the yield load or ultimate load is the critical stress multiplied by the area of cross section.

that is $P_{ult} = f_{cr} \times A$ which is rearranged as

$$\frac{P_{ult}}{A} = \frac{f_{cr}}{F_y} \cdot F_y = \frac{1 \cdot 67 f_c}{1 \cdot 67 f_b} \cdot F_y = \frac{f_c}{f_b} F_y$$

Where f_c and f_b are the allowable compressive and bending stresses respectively comparing with the expression

Yield stress $= Q.F_y$

$$Q = \frac{f_c}{f_b} \qquad \text{...... (36)}$$

c) *For members consisting of both stiffened and unstiffened elements* — The member consisting of both stiffened and unstiffened elements will attain its failure load when the weaker of the unstiffened elements buckles at the critical stress. At the stress A_{eff} will consist of unreduced area of unstiffened elements and effective area of the stiffened elements computed for f_{cr}.

$P_{ult} = f_{cr} \times A_{eff}$ which is rearranged as

$$\frac{P_{ult}}{A} = \frac{f_{cr}}{F_y} \cdot \frac{A_{eff}}{A} \cdot F_y = \frac{f_c}{f_b} \cdot \frac{A_{eff}}{A} \cdot F_y$$

Yield stress $= Q \cdot F_y$

$$Q = \frac{f_{cr}}{F_y} \cdot \left(= \frac{f_c}{f_b} \right) \times \frac{A_{eff}}{A} \qquad \ldots\ldots (37)$$

That is product of $Q_{unstiffened} \times Q_{stiffened}$

9.4.2.2 *The allowable stress in axial compression*

a) *Factor of safety* — The factor of safety for compression members is taken as 1·92 which is about 15 percent larger than the basic safety factor of 1·67 used in most part of the specification. This increase is to compensate for the greater sensitivity of the compression members to accidental imperfections of the shape or accidental load eccentricities.

The expressions for the compression stress in the elastic range is based on Euler critical stress $f_{cr} = \frac{\pi^2 E}{(KL/r)^2}$ where K is the effective length factor. For the inelastic range a parabolic variation is assumed. The limit of inelastic buckling is taken as 0·5 F_y. As the cold worked members have residual stresses the limit of proportionality assumed 0·5 F_y. For light gauge members the effective $F_y = Q.F_y$ and hence the limit of slenderness ratio at which elastic buckling starts is obtained as

$$0{\cdot}5\, Q.F_y = \frac{\pi^2 E}{(KL/r)^2}$$

$$\text{or } (KL/r)^2 = \frac{\pi^2 E}{0{\cdot}5\, F_y.Q} = \frac{2\pi^2 E}{Q.F_y} \qquad \ldots\ldots (38)$$

$$\text{Therefore } \left(\frac{KL}{r}\right)_{limit} = \sqrt{\frac{2\pi^2 E}{Q.F_y}}$$

This limit is denoted by the symbol $\frac{C_c}{\sqrt{Q}}$, where $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$

The expression in the inelastic range is obtained by taking the equation for parabola as

$$f = Q{\cdot}F_y\left[A - \frac{1}{B}\cdot\frac{QF_y}{f_{cr}}\right]$$

between the limit at $\frac{KL}{r} = 0$; $f = F_y$ and

at $KL/r = \frac{C_0}{\sqrt{Q}} = \sqrt{\frac{2\pi^2E}{Q{\cdot}F_y}}$; $f = 0{\cdot}5\ F_y$

By substituting these two conditions, the value of $A = 1$ and $B = 4$,

$$\therefore f = Q{\cdot}F_y\left[1 - \frac{1}{4}\,\frac{Q{\cdot}F_y}{\dfrac{\pi^2E}{(KL/r)^2}}\right] \qquad \text{......} \ (39)$$

The expressions are:

1) for inelastic range $f = Q{\cdot}F_y \ 1 - \left[\frac{Q{\cdot}F_y}{4\pi^2 E}\left(\frac{KL}{r}\right)^2\right]$

2) for elastic range $f = \dfrac{\pi^2E}{\left(\dfrac{KL}{r}\right)^2}$

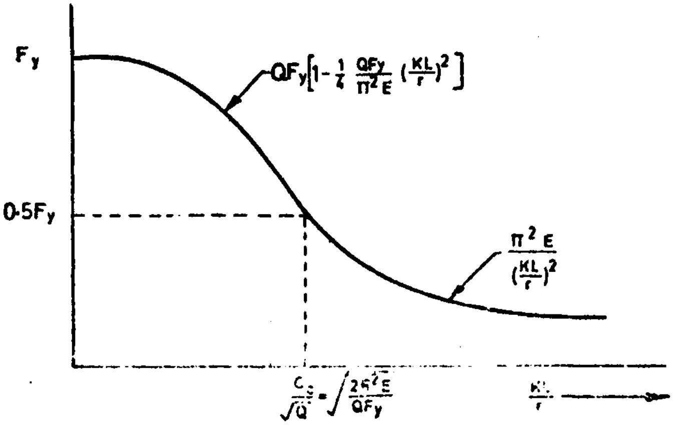


FIG. 17

The allowable stresses are obtained by dividing the above expressions by the factor of safety 1·92 that is for $KL/r < C_0/\sqrt{Q}$:

$$F_a = \frac{12\,Q \cdot F_y}{23} - \frac{3}{23}\,\frac{(Q \cdot F_y)^2}{\pi^2 E} \cdot \left(\frac{KL}{r}\right)^2 \qquad \text{...... (40)}$$

For $KL/r > C_0/\sqrt{Q}$, $F_a = \dfrac{12}{23}\,\dfrac{\pi^2 E}{(KL/r)^2}$

By substituting $E = 2\ 074\ 000$ kgf/cm^2,

$$F_a = \frac{10\ 680\ 000}{(KL/r)^2} \qquad \text{...... (41)}$$

b) When the factor Q is equal to unity, the steel is 2·29 mm or more in thickness and KL/r is less than C_0, the factor of safety is taken as equal to that of a hot-rolled section. The factor of safety varies as a quarter of sine curve with $KL/r = 0$ it is 1·67 and becomes 1·92 for $KL/r = C_0$.

When the factor $Q = 1$, the expression (39) can be written as

$$f = F_y \left[1 - \frac{1}{4}\,\frac{F_y}{\dfrac{\pi^2 E}{(KL/r)^2}} \right]$$

As $C^2_0 = \dfrac{2\pi^2 E}{F_y}$

$$f = F_y \left[1 - \frac{1}{2C^2_0}\,(KL/r)^2 \right] \qquad \text{...... (42)}$$

The factor of safety for thicker members where the local buckling is not existing, is taken as equal to that for hot-rolled sections, that is, a value of 1·67 for $KL/r = 0$ to a value of 1·92 for $KL/r = C_0$ and between these values the variation of the factor of safety is a sine function. The expression for factor of safety is

$$F.S. = \frac{5}{3} + \frac{3}{8}\,(KL/r)/C_0 - \frac{1}{8}\left[(KL/r)/C_0\right]^3 \qquad \text{...... (43)}$$

Hence in **6.6.1.1**(b) of IS : 801-1975 that is for members with $Q = 1$ and thickness of member if more than 2·29 mm the allowable stress for axial compression is obtained by dividing the expression (42) by (43).

9.5 Combined Axial and Bending Stress — If in a member initially there is a deviation from the straightness or deflection or eccentricity from whatever cause, the application of an axial force causes this deflection or eccentricity to be magnified in the ratio $\dfrac{1}{1-\dfrac{P_A}{P_E}}$ which is known as the magnification factor. This additional deflection causes additional bending moment. Hence the interaction formula for such cases is

$$\frac{f_a}{F_a}+\frac{f_{bx}}{F_{bx}}\,\frac{C_m}{\left(1-\dfrac{f_a}{F'_{ex}}\right)}<1\cdot0$$

the factor $\left(\dfrac{1}{1-\dfrac{P_A}{P_E}}\right)$ can also be written as $\left(\dfrac{1}{1-\dfrac{f_a}{F_E}}\right)$

where P_E is the Eulers load and F_E is the Euler stress. If the bending moment is acting about both the axes then the chord term also enters.

namely $\dfrac{f_{by}}{\left[1-\dfrac{f_a}{F'_{ey}}\right]F_{by}}$

For smaller axial loads that is $\dfrac{f_a}{F_a}<0\cdot15$, the term $\dfrac{f_a}{F_e}$ is very small compared to 1 and hence the magnification factor is taken as 1 itself. These give the expressions given under **6.7** of IS : 801-1975. C_m is a coefficient to take into consideration the end moments in the members.

10. WALL STUDS

10.1 Cold-formed steel studs in walls or load carrying partitions are often employed in a manner different from that used in heavy steel framing, but similar to that used in timber construction of residential buildings. Such studs are faced on both sides by a variety of wall material such as fibreboard, pulp board, plywood and gypsum board. While it is the main function of such wall sheathing to constitute the actual outer and inner wall surfaces and to provide the necessary insulation, they also serve as bracing for the wall studs. The latter, usually of simple or modified I or channel shape with webs placed perpendicular to the wall surface, would buckle about their minor axes, that is, in the direction of the wall at prohibitively low loads. They are prevented from doing so by the lateral restraint against deflection in the direction of the wall provided by the wall sheathing. If the lateral support is correctly designed, such studs, if loaded to destructions will fail buckling out of the wall, the corresponding buckling load obviously represents the highest load which the stud may reach. The wall sheathing therefore contributes to the structural economy by maximising the usable strength of the stud.

10.2 The necessary requirements in order to assure that the wall sheathing provide the lateral support necessary for the described optimum functioning of the studs are stipulated in **8.1** of IS : 801-1975. In order that collateral wall material furnish the support to the studs to which it is attached, the assembly (studs, wall sheathing, and the connections between the two) shall satisfy the following three conditions:

a) The spacing between attachments (screws, nails, clips, etc) shall be close enough to prevent the stud from buckling in the direction of the wall between attachments.

b) The wall material shall be rigid enough to minimise deflection of the studs in the direction of the wall which, if excessive, could lead to failure in one of the two ways, namely, (1) the entire stud could buckle in the direction of the wall in a manner which would carry the wall material with it, and (2) it could fail simply by being overstressed in bending due to excessive lateral deflection.

c) The strength of the connection between wall material and stud must be sufficient to develop a lateral force capable of resisting the buckling tendency of the stud without failure of the attachment proper by tearing, loosening, or otherwise.

10.2.1 The first of these conditions is satisfied by **8.2**(b) of IS : 801-1975. This stipulates that the slenderness ratio a/r_2 for minor-axis buckling between attachments (that is, in the direction of the wall) shall not exceed one-half of the slenderness ratio L/r_1 for major-axis buckling, that is, out of the wall. This means that with proper functioning of attachments buckling out of the wall will always occur at a load considerably below that which would cause the stud to buckle laterally between attachments. Even in the unlikely case if an attachment was defective to a degree which would make it completely ineffective, the buckling load would still be the same for both directions (that is, $a/r_2 = L/r_1$), so that premature buckling between attachments would not occur.

10.2.2 In regard to conditions (b) the rigidity of the wall material plus attachments is expressed as its modulus of elastic support k that is, the ratio of the applied force to the stretch produced by it in the sheathing-attachment assembly.

The minimum modulus k which shall be furnished by the collateral material in order to satisfy condition (b), above, that is, to prevent excessive 'buckling' of the stud in the direction of the wall. It defines the minimum rigidity (or modulus k) which is required to prevent from lateral buckling a stud which is loaded by $P = A.F_y$, that is, stressed right up to the yield point of the steel.

It may be seen from **8.1**(c) of IS : 801-1975 that the required modulus of support *k* is directly proportional to the spacing of attachments *a*.

10.2.3 It remains to satisfy condition (c) above to the effect that the strength of the attachment of wall material to stud shall be sufficient so that it will not give way at a load on the stud which is smaller than its carrying capacity. This is achieved by means of provision (d) of **8.1** of IS : 801-1975.

10.3 Theory indicates that an ideal (straight, concentric) stud which is elastically supported at intermediate points (such as by wall attachments) will not exert any force on these attachments until it reaches its buckling load. In contrast, analysis and test indicate that intermediately supported ' real ', that is, imperfect studs (crooked, eccentric) do exert pressure on their support increasingly so that the load on the stud is increased.

It may be noted that a value *L*/240 has been provided to allow for the imperfections.

11. CHANNEL AND Z-BEAMS

11.1 Among hot-rolled sections, I-shapes are most favourable for use as beams because a large portion of the material is located in the flanges, at the maximum distance from the axis. In cold-formed construction, the only two-flange shapes which may be formed of one single sheet (without welding or other connection) are the channel, the Z-shape, and the hat. Of these, the hat shape has the advantage of symmetry about the vertical axis and of great lateral stability; its use is correspondingly separate webs which pose problems of access, connection, etc.

Channels and Z-shapes are widely used. Neither of them is symmetrical about a vertical plane. Since, in most applications, loads plate is applied in the plane of the web, lack of symmetry about that plane calls for special measures to forestall structurally undesirable performance (lateral deflection, twisting, etc). Appropriate provisions for this purpose are contained in IS : 801-1975.

11.2 Connecting Two Channels to Form an I-Beam

11.2.1 There are various ways of connecting two or more cold-formed shapes to produce an I-section. One of these is by spot-welding an angle to each flange of channel (*see* shapes 15 and 17 of Fig. 1). Another is to connect two channels back to back by two rows of spot-welds (or other connectors) located as closely as possible to top and bottom flanges. The shapes 14 and 16 of Fig. 1 are sections of this sort. Provisions for the correct proportioning of the connecting welds for such shapes are given in 7.3 of IS : 801-1975.

11.2.2 In view of lack of symmetry or anti-symmetry about a vertical plane the so-called shear-centre of a channel is neither coincident with the centroid (as it is in symmetrical or anti-symmetrical shapes) nor is it located in the plane of the web. The shear-centre is that point in the plane of a beam section through which a transverse load should act in order to produce bending without twisting. In a channel the shear centre is located at a distance m back of the midplane of the web, as shown in Fig. 18. The distance m for channels with and without flange lips is given in 7.3 of IS : 801-1975. The internal shear force V passes through this point. Consequently, if the external load P was applied at the same point (such as by means of the dotted bracket in Fig. 18) the two forces would be in line and simple bending would result. Since loads in most cases actually act in the plane of the web, each such load produces a twisting moment P_m, unless these torques are balanced by some externally applied counter-torques, undesirable twisting will result.

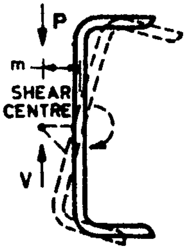


FIG. 18 SHEAR CENTRE OF A CHANNEL

11.2.3 If two channels are joined to form an I-beam, as shown on Fig. 19A each of them is in the situation shown on Fig. 19B and tends to rotate in the sense indicated by the arrow on that figure. The channels, then, tend through rotation to separate along the top, but this tendency is counteracted by the forces in the welds joining them. These forces S_w, constitute an opposing couple; they are shown on Fig. 19B, which represents a short portion of the right channel, of length equal to the weld spacings. This portion, delimited by dotted lines on Fig. 19A, contains a single pair of welds, and P is the total force acting on that piece of one channel, that is half the total beam load over the lengths. From the equality of moments:

$$P_m = S_w C, \text{ so that } S_w = P (m/c) \qquad \text{...... (44)}$$

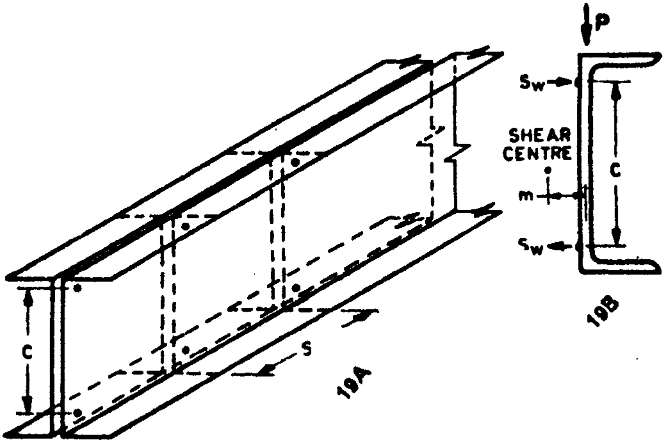


FIG. 19 CHANNELS SPOT WELDED TO FABRICATE I-BEAM

11.2.3.1 It is seen that the weld force S_w depends upon the load acting in the particular longitudinal spacing between welds S. If P is the intensity of load on the beam at the location of the particular weld, the load on the channel is $P = \frac{ps}{2}$.

Substituting this in equation (44) we have the required weld strength $S_w = \frac{mps}{2c}$ ...... (45)

where

S_w = required strength of weld,

s = longitudinal spacing of welds,

c = vertical distance between two rows of welds near or at top and bottom flanges,

p = intensity of load per unit length of beam, and

m = distance of the shear centre from middle plane of the web of channel.

11.2.4 It is seen that the required weld strength depends on the local intensity of load on the beam at that weld. Beams designed for 'uniform load' actually are usually subjected to more or less uneven load, such as from furniture and occupants. It is, therefore, specified that for 'uniformly loaded beams' the local load intensity P shall be taken as three times the uniform design load. 'Concentrated' loads or reactions p are actually distributed over some bearing length B; if B is larger than the weld spacing s, than the local intensity is obviously p/B. If, on the other hand, the bearing length is smaller than the weld spacing, then the pair of welds nearest to the load or reaction shall resist the entire torque $(P/2)m$, so that $S_w = Pm/2c$. Since the main formula above is written in terms of a load intensity p, it is convenient to use an equivalent intensity for this case which is $p = P/2s$; the correctness is easily checked by substituting this value in the general equation 44 (*see also* 7.3 of IS : 801-1975).

11.3 Bracing of Single-Channel Beams

11.3.1 If channels are used singly as beams, rather than being paired to form I-sections, they should evidently be braced at intervals so as to prevent them from rotating in the manner indicated in Fig. 18. For simplicity, Fig. 20 shows two channels braced at intervals against each other. The situation is evidently much the same as in the composite I-section of Fig. 19A, except that the role of the welds is now played by the braces. The difference is that the two channels are not in contact, and that the spacing of braces is generally considerably larger than the weld spacing.

In consequence, each channel will actually rotate very slightly between braces, and this will cause some additional stresses which superpose on the usual simple bending stresses. Bracing shall be so arranged that (a) these additional stresses are sufficiently small so that they will not reduce the carring capacity of the channel (as compared to what it would be in the continuously braced condition) , and (b) rotations are kept small enough to be unobjectionable (for example, in regard to connecting other portions of the structure to the channels), that is, of the order of 1 to 2°.

11.3.1.1 Corresponding experimental and analytical investigations have shown that the above requirements are satisfied for most distributions of beam loads, if between supports not less than three equidistant braces are placed (that is, at quarter-points of the span or closer). The exception is the case where a large part of the total load of the beam is concentrated over a short portion of the span; in this case an additional brace should be placed at such a load. Correspondingly, **7.3** of IS : 801-1975 stipulates that the distance between braces shall not be greater than one-quarter of the span; it also defines the conditions under which an additional brace should be placed at a load concentration.

11.3.2 For such braces to be effective it is not only necessary that their spacing is appropriately limited but also that strength is suffice to provide

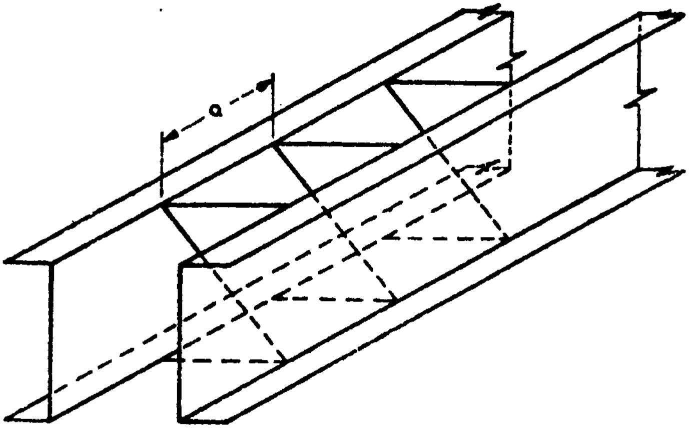


FIG. 20 BRACED CHANNELS

the force necessary to prevent the channel from rotating. It is, therefore, necessary also to determine the forces which will act in braces such as shown in Fig. 21A. These forces are found if one considers (as shown in the figure) that the action of a load applied in the plane of the web (which causes a torque Pm) is equivalent to that same load when applied at the shear centre (where it causes no torque) plus two forces $f = Pm/h$ which, together, produce the same torque Pm. As is sketched in Fig. 21B, each half of the channel may then be regarded as a continuous beam loaded by the horizontal forces f and supported at the brace points. The horizontal brace force is then, simply, the appropriate reaction of this continuous beam. The provisions of 8.2.2 of IS : 801-1975 represent a simple and conservative approximation for determining these reactions, which are equal to the force P_b which the brace is required to resist at each flange.

11.4 Bracing of Z-Beams — Most Z-sections are anti-symmetrical about the vertical and horizontal centroidal axes. In view of this the centroid and the shear centre coincide and are located at the mid-point of the web. A load applied in the plane of the web has no lever arm about the shear centre ($m = 0$) and does not tend to produce the kind of rotation a similar load would produce on a channel. However, in Z-sections the principal axes are oblique to the web (Fig. 22). A load applied in the plane of the web, resolved in the direction of the two axes, produces deflections in each of them. By projecting these deflections into the

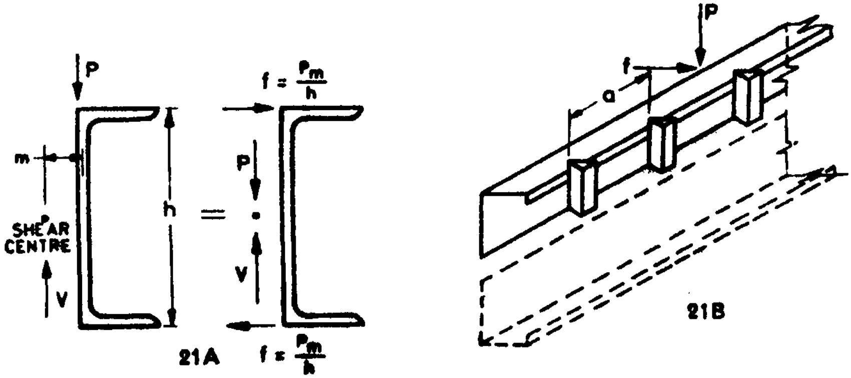


Fig. 21 Load Acting at the Shear Centre — Its Effects on the Braces

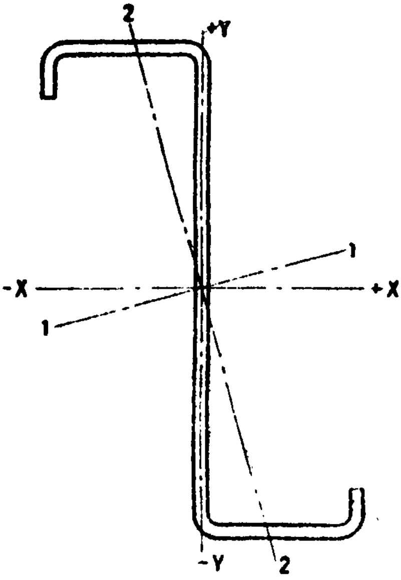


Fig. 22 Principal Axes in Z-Sections

horizontal and vertical planes it is found that a Z-beam loaded vertically in the plane of the web deflects not only vertically but also horizontally. If such deflection is permitted to occur then the loads moving sideways with the beam, are no longer in the same plane with the reactions at the ends. In consequence, the loads produce a twisting moment about the line connecting the reactions. In this manner it is seen that a Z-beam, unbraced between ends and loaded in the plane of the web, deflects laterally and also twists. Not only are these deformations likely to interfere with a proper functioning of the beam, but the additional stresses caused by them produce failure at a load considerably lower than when the same beam is used fully braced. Appropriate experimental and analytical investigation has shown that intermittently braced Z-beams may be analysed in much the same way as intermittently braced channels. It is merely necessary, at the point of each actual vertical load P, to apply a fictitious load f. It is in this manner that the provisions applicable to bracing of Z-shaped beams in **8.2** of IS : 801-1975 have been arrived at.

NOTE — Since Z-shapes and channels are the simplest two-flange sections which can be produced by cold-forming, one is naturally inclined to use them as beams loaded in the plane of the web. However, in view of their lack of symmetry, such beams require special measures to prevent tipping at the supports, as well as relatively heavy braking to counteract lateral deflection and twisting in the span. Their use is indicated chiefly where continuous bracing exists, such as when they are incorporated in a rigid floor or roof system, so that special intermittent braking may be required during erection only. For such erection condition in **8.2** of IS : 801-1975 may be chiefly useful. For conditions other than these, serious consideration should be given to hat sections. These have the same advantages as channel and Z-sections (two-flange section produced by simple cold-forming) but none of their disadvantages. They are, in fact, in some respects superior to I-sections.

12. CONNECTIONS

12.1 General — A considerable variety of means of connection finds application in cold-formed construction. Without any claim for completeness, these may be listed as follows:

a) *Welding* — which may be sub-divided into resistance welding, mostly for shop fabrication, and fusion welding, mostly for erection welding;

b) *Bolting* — which may be sub-divided into the use of ordinary ' black ' bolts without special control on bolt tension, and the use of high-strength bolts with controlled, high bolt tension;

c) *Riveting* — while hot riveting has little application in light-gauge construction, cold-riveting finds considerable use, particularly in special forms, such as blind rivets (for application from one side only), tubular rivets (to increase bearing area), high shear rivets, explosive rivets, and others;

d) *Screwing* — mostly by means of self-tapping screws of a considerable variety of shapes; and

e) *Special devices* — among which may be mentioned; (1) metal stitching, achieved by tools which are special developments of the common office stapler, and (2) connecting by upsetting, by means of special clinching tools which draws the sheets into interlocking projections.

12.1.1 Provisions only for welding and for black bolts are contained in IS : 800-1962. Information on high strength, high-tensioned bolts is available and will be briefly discussed herein. Classes (c), (d) and (e) in **12.1**, above, mostly refer to a variety of proprietary devices in regard to which information on strength of connections shall be obtained from manufacturers (preferably based on tests preformed by independent agencies), or from tests carried out by or for the prospective user. In regard to riveting and, to a lesser extent, screwing, the data given in the code in regard to bolting may be used as a general guide.

12.2 Welding

12.2.1 *Spot Welding* — In its normal form as well as by projection, welding is probably the most important means of shop connection in light-gauge steel fabrication. Welding procedure and design strength of spot welds are specified in IS : 819-1957. Welding procedure specifications contain definite recommendations on electrode diameter, current, etc, depending on sheet thickness. The use of the design values of IS : 819-1957 is, therefore, justified only if the specified welding procedures are strictly followed.

12.2.2 *Fusion Welding* — It is used for connecting cold-formed light-gauge steel members to each other as well as connecting such members to heavy hot-rolled steel framing (such as floor panels or floor joists to beams and girders of the steel frame). It is used in fillet welds, butt welds (rather rarely), and in plug or puddle welds. These latter are often used in connecting light-gauge to heavy rolled steel and are made by burning a circular hole through the sheet and fillet-welding the sheet along the periphery of the hole to the underlying, heavy steel section.

12.2.2.1 The allowable stresses for fusion welds are given in **7.2** of IS : 801-1975. It is mentioned that shear stresses are referred to 'the throat' of the weld. This throat is a fictitious dimension, equal to 0·707 t (t being the sheet thickness), the meaning of which is shown in Fig. 23, that is, in welding thin sheet the weld shape generally obtained is that shown on the figure, with the thickness of the weld actually exceeding that of the sheet. The intention is to disregard any material deposited beyond the dashed line in Fig. 23, and to calculate the throat thickness in the same manner as in heavy welded construction.

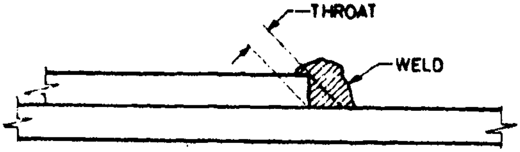


FIG. 23 THROAT OF A FILLET WELD

12.2.2.2 When plug welds are made with pre-punched holes, the length of the fillet weld for computing weld strength is identical with the perimeter of the hole. When the hole is burned and the weld made in the same operation, a frequent process (which is more aptly designated as puddle-welding), a conservative procedure is to compute the perimeter for a hole of diameter 6 to 10 mm less than the visible diameter of the puddle.

12.2.2.3 It should be added that the welding of thin steel sheet requires a high degree of skill and welding technique. Welders who have successfully passed the usual proficiency tests for welding of heavy sections, as a rule, are not capable without special additional training and experience to produce satisfactory welds of light-gauge members. Moreover, the welding together of two sections of radically different thicknesses, such as the welding of light-gauge panels or joists to ordinary, heavy steel beams or girders, again requires special techniques. A well-trained, skilled welder usually will acquire and develop these special techniques with a reasonable amount of practice, but such practice should be acquired not on the job, but in advance on special practice welds, and under competent supervision.

12.3 Bolting

12.3.1 *Black Bolts in Ordinary Connections* — The nature of light-gauge, cold-formed construction generally precludes the use of turned and fitted both. The provisions of 7.5 of IS : 801-1975 therefore, are written for black bolts in oversize holes (usually 1·5 mm oversize for bolts of 12 mm diameter and larger, and 0·75 mm for smaller bolts).

These provisions of safeguard against the following four types of failure observed in tests, generally with a safety factor of the order of 2·5, which was selected in view of the significant scatter in these tests.

12.3.2 *High Tensile Friction Grip Bolts*

12.3.2.1 The use of such bolts for connections in hot-rolled steel work has become very common in a number of countries. Such connections differ in two respects from those made with ordinary black bolts:

a) the material from which these bolts are made has about twice the tensile strength of ordinary, black bolts; and

b) the nuts of such bolts are torqued to prescribed amounts which result in a minimum bolt tension of 90 percent of the proof load of the bolt (the proof load is about equal to the proportional limit of the bolt).

One of the chief advantages of these connections is that they eliminate connection slip which would occur if these same connections were made with unfinished black bolts. They also increase the shear strength of the connection (*see* IS : 4000-1967).

12.3.2.2 In order to investigate the possible advantages in the field of light-gauge steel construction of using high-strength bolts with controlled high bolt tension, a number of tests have been made on connections of this type, with the bolts and bolt tensions (torques) complied with the regulations governing the use of such bolts in heavy steel construction as in IS : 4000-1967.

12.3.2.3 It has also been found in these tests that the use of high tensioned bolts will effectively eliminate connection slip at design loads regardless of whether the faying surfaces are bare, painted, or galvanized. This may be of importance in situations where small deformations in connections may cause relatively large distortions of the structure, such as in knee-braces of portal frames, in rigid joint construction generally, and in many other situations of the like.

12.3.2.4 This brief summary will indicate the economic possibilities of high strength bolting in light-gauge construction. These may be utilized only if special bolts are available, and special assembly techniques are strictly adhered to, such as specified in the quoted specifications.

12.4 Spacing of Connection in Compression Elements — If compression elements are joined to other parts of the cross section by intermittent connections, such as spot welds, these connections shall be sufficiently closely spaced to develop the required strength of the connected element. For instance, if a hat section is converted into a box shape by spot welding a flat plate to it, and if this member is used as a beam with the flat plate up, that is in compression (*see* Fig. 24), then the welds along both lips of the hat should be placed so as to make the flat plate act monolithically with the hat. If welds are appropriately, spaced, this flat plate will act as a 'stiffened compression element' with width *w* equal to distance between rows of welds, and the section can be calculated accordingly.

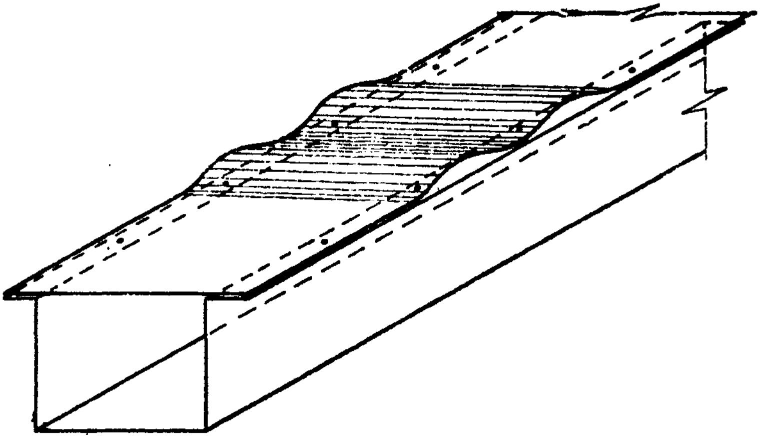


FIG. 24 PLATE SPOT-WELDED TO HAT SECTION

13. MISCELLANEOUS

13.1 Usually Wide, Stable Beam Flanges — Compression flanges of large *w/t* ratio tend to lose their stability through buckling. However, if flanges are unusually wide they may require special consideration even if there is no tendency to buckling, such as in tension flanges. Two matters need consideration for such elements; shear lag, which depends on the span-width ratio and is independent of the thickness, and curling which is independent of the span and does depend on the thickness.

13.2 Shear Lag — In metal beams of the usual shapes, the normal stresses are induced in the flanges through shear stresses transferred from the web to the flange. These shear stresses produce shear strains in the flange which, for ordinary dimensions, have negligible effects. However, if flanges are unusually wide (relative to their length) these shear strains have the effect that the normal bending stresses in the flanges decrease with increasing distance from the web. This phenomenon is known as shear lag. It results in a non-uniform stress distribution across the width of the flange, similar to that in stiffened compression elements, though for entirely different reasons. As in the latter case, the simplest way of accounting for this stress variation in design is to replace the non-uniformly stressed flange of actual width *w* by one of reduced, effective width subject to uniform stress.

13.3 Flange Curling

13.3.1 In beams which have unusually wide and thin, but stable flanges (that is, primarily tension flanges with large *w/t* ratios), there is a tendency for these flanges to curl under load. That is, the portions of these flanges most remote from the web (edges of I-beams, centre portions of flanges of box or hat beams) tend to deflect towards the neutral axis.

13.3.2 In **5.2.3** of IS : 801-1975, there is given an approximate formula which permits one to compute the maximum admissible flange width W_{max} for a given amount of tolerable curling *c*.

It will be noted that **5.2.3** of IS : 801-1975 does not stipulate the amount of curling which may be regarded as tolerable, but merely suggests in the foot note that an amount equal to about 5 percent of the depth of the section is not excessive under usual conditions. It will be found that the cases are relatively rare in which curling becomes a significant factor in limiting flange width, except where, for the sake of appearance, it is essential to closely control out-of-plane distortions (for example, when flat ceilings are to be formed of wide, cellular floor or roof panels).

13.4 Application of Plastic Design to Light-Gauge Structures

13.4.1 Considerable research and development effort is under way in the field of steel structures for buildings to develop plastic design methods. Within certain limits these methods are at present admitted in design codes as optional alternatives to conventional (either ' simple ', ' semi-rigid ' or ' rigid ') design methods (*see*, for example, **14.2** of IS : 800-1962).

13.4.2 Plastic design is based on the proven proposition that a mild steel beam does not fail when the yield stress is reached in the outer fibre. It continues to function, and gives way through excessive deformation only when yielding has practically reached the neutral axis from both sides, thus forming a ' yield hinge '. In continuous structures yield hinges form successively and produce a redistribution of moments which generally permits a more economical design. Failure occurs only when enough hinges have formed to convert the structure (rigid frame, continuous beam, etc) into a mechanism.

13.4.3 This process requires that all hinges, except the last, be capable of undergoing rotations, often considerable, while the steel in practically the entire section is yielding, at the same time. Compact sections are capable of performing in this manner. However, many compression flanges even if they are rigid enough (reasonably small *w/t* ratio) not to buckle immediately when the stress reaches the yield point, will buckle very shortly thereafter if submitted to further compression strain, such as would be caused by the rotation of plastic hinges. It has been established in recent research at Lehigh University and elsewhere that in order for a flange section to perform satisfactorily in connection with plastic design, limitations shall be imposed on w/t and h/t ratios which are significantly more stringent than those in use in conventional design. If this is not done, members at plastic hinges will prematuraly buckle locally and the carrying capacity computed by plastic methods will not be reached.

13.4.4 It is evident from this that most shapes now in use in light-gauge steel structures, since they have w/t ratios considerably in excess of conventional hot-rolled shapes, are not capable of developing plastic hinges satisfactorily and of maintaining them throughout the required rotations without local buckling. It follows that plastic design methods are not applicable to light-gauge construction in its present form, unless such construction is surrounded with additional safeguards of the kind which are now in the process of development for hot-rolled structures. What is more, it is obvious that those shapes most typical of light-gauge steel, such as panels and decks, by their very nature require large w/t ratios which preclude satisfactory performance under plastic design conditions. This is not to say that, through appropriate research and development, cold-formed sections suitable for structural framing (as distinct from panels and decks) could not be developed with sufficient section stability to be amenable to plastic design.

13.4.5 It should be noted that these reservations apply to the full development of plastic hinges. There are a number of unsymmetrical sections in light-gauge steel construction, such as many roof decks, where the neutral axis is much closer to the compression than to the tension flange. In such sections the (stable) tension flange yields first, but failure does not occur at that load at which such yielding begins. Only when yielding has spread over much of the section, including the compression side will the member fail at a load considerably higher than that which initiated tension yielding. This development has been used as early as 1946 for the successful interpretation of tests on stiffened compression elements. It is this ability of unsymmetrical sections to redistribute their stresses through plastic action which accounts for the excess of their strength over and above that computed on the conventional, elastic basis. In such more limited connections plastic analysis is needed for a full understanding of structural performance even of some thin-wall sections.

13.4.6 However, since light-gauge structures of the presently current types (a) usually have compression flanges too thin to develop plastic hinges without local buckling, and (b) are usually not of the continuous-beam or rigid-frame type; the application of plastic design to light-gauge steel structures is much more restricted and of less consequence than in hot-rolled construction.

SECTION 2 DESIGN TABLES AND DESIGN CURVES

1. SCOPE

1.1 This section contains various design tables and design curves, required in design of structures using light-gauge steel sections in accordance with the provisions of IS : 801-1975.

2. DESIGN OF STIFFENED COMPRESSION ELEMENTS — ELEMENTS WITHOUT INTERMEDIATE STIFFENERS

2.1 The limiting width thickness ratio (w/t_{lim}) for compression elements below which the element is fully effective ($b = w$) has been tabulated in Table 1 both for nontubular and tubular section for the load and deflection determination. The values of w/t_{lim} have been calculated in accordance with the formulae contained in **5.2.1.1** of IS : 801-1975.

TABLE 1 STIFFENED COMPRESSION ELEMENTS LIMITING WIDTH THICKNESS RATIO w/t_{lim} BELOW WHICH ELEMENT IS FULLY EFFECTIVE

Stress in Compression Element f, kgf/cm^2	Nontubular Section		Tubular Section	
	For Load Determination	For Deflection Determination	For Load Determination	For Deflection Determination
100	143·5	185·00	154·0	199·0
200	101·4	130·72	108·80	140·71
300	82·85	106·81	88·91	114·89
400	71·75	92·50	77·00	99·5
500	64·18	82·74	68·87	89·0
600	58·58	75·52	62·86	81·24
700	54·24	69·92	58·21	75·21
800	50·73	65·40	54·44	70·36
900	47·83	61·66	51·33	66·33
1 000	45·38	58·50	48·70	62·93
1 100	43·27	55·78	46·43	60·0
1 200	41·42	53·40	44·45	57·45
1 300	39·8	51·31	42·71	55·19
1 400	38·35	49·44	41·15	53·18
1 500	37·05	47·76	39·76	51·38
1 600	35·88	46·25	38·50	49·75
1 700	34·80	44·86	37·34	48·26
1 800	33·82	43·60	36·29	46·90
1 900	32·92	42·44	35·33	45·65
2 000	32·09	41·37	34·44	44·5
2 100	31·31	40·36	33·60	43·43

2.2 Nontubular Section — Design curves, worked out in accordance with the provisions of 5.2.1.1 of IS : 801-1975, for the load and deflection determination of nontubular members have been given in Fig. 25A, 25B, 26A and 26B.

2.3 Tubular Sections — Design curves, worked out in accordance with the provisions of **5.2.1.1** of IS : 801-1975, for the load and deflection determination of tubular member have been given in Fig. 27A, 27B, 28A and 28B.

3. DESIGN OF STIFFENED COMPRESSION ELEMENTS — MULTIPLE STIFFENED ELEMENTS AND WIDE STIFFENED ELEMENTS WITH EDGE STIFFENERS

3.1 Values of α, the reduction factor for computing the effective area of stiffeners, as contained in **5.2.1.2** of IS : 801-1975 are given in Table 2 for w/t ratio between 60 and 150.

TABLE 2 REDUCTION FACTOR, α, FOR COMPUTING EFFECTIVE AREA OF STIFFENERS ($A_{ef} = \alpha A_{st}$)

For $60 < w/t < 90$,

$$\alpha = (3 - 2\,b_e/w) - \frac{1}{30}\left[1 - \frac{b_e}{w}\right](w/t)$$

For $\frac{w}{t} \geqslant 90$

$$\alpha = b_e/w$$

w/t	FLAT WIDTH RATIO, b/t												
	20	25	30	35	40	45	50	55	60	70	80	90	100
60	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00
62	0·95	0·96	0·97	0·97	0·98	0·98	0·99	0·99	1·00	1·00	1·00	1·00	1·00
64	0·91	0·92	0·93	0·94	0·95	0·96	0·97	0·98	0·99	1·00	1·00	1.00	1·00
66	0·86	0·88	0·89	0·91	0·92	0·94	0·95	0·97	0·98	1·00	1·00	1·00	1·00
68	0·81	0·83	0·85	0·87	0·89	0·91	0·93	0·95	0·97	1·00	1·00	1·00	1·00
70	0·76	0·79	0·81	0·83	0·86	0·88	0·90	0·93	0·95	1·00	1·00	1·00	1·00
72	0·71	0·74	0·77	0·79	0·82	0·85	0·88	0·91	0·93	0·99	1·00	1·00	1·00
74	0·66	0·69	0·72	0·75	0·79	0·82	0·85	0·88	0·91	0·97	1·00	1·00	1·00
76	0·61	0·64	0·68	0·71	0·75	0·78	0·82	0·85	0·89	0·96	1·00	1·00	1·00
78	0·55	0·59	0·63	0·67	0·71	0·75	0·78	0·82	0·86	0·94	1·00	1·00	1·00
80	0·50	0·54	0·58	0·63	0·67	0·71	0·75	0·79	0·83	0·92	1·00	1·00	1·00
82	0·45	0·49	0·53	0·58	0·62	0·67	0·71	0·76	0·80	0·89	0·98	1·00	1·00
84	0·39	0·44	0·49	0·53	0·58	0·63	0·68	0·72	0·77	0·87	0·95	1·00	1·00
86	0·33	0·39	0·44	0·49	0·54	0·59	0·64	0·69	0·74	0·84	0·94	1·00	1·00
88	0·28	0·33	0·38	0·44	0·49	0·54	0·60	0·65	0·70	0·81	0·92	1·00	1·00
90	0·22	0·28	0·33	0·39	0·44	0·50	0·56	0·61	0·67	0·78	0·89	1·00	1·00
100	0·20	0·25	0·30	0·35	0·40	0·45	0·50	0·55	0·60	0·70	0·80	0·90	1·00
110	0·18	0·23	0·27	0·32	0·36	0·41	0·45	0·50	0·55	0·64	0·73	0·82	0·91
120	0·17	0·21	0·25	0·29	0·33	0·38	0·42	0·46	0·50	0·58	0·67	0·75	0·83
130	0·15	0·19	0·23	0·27	0·31	0·35	0·38	0·42	0·46	0·54	0·62	0·69	0·77
140	0·14	0·18	0·21	0·25	0·29	0·32	0·36	0·39	0·43	0·50	0·57	0·64	0·71
150	0·13	0·17	0·20	0·23	0·27	0·30	0·33	0·37	0·40	0·47	0·53	0·60	0·67

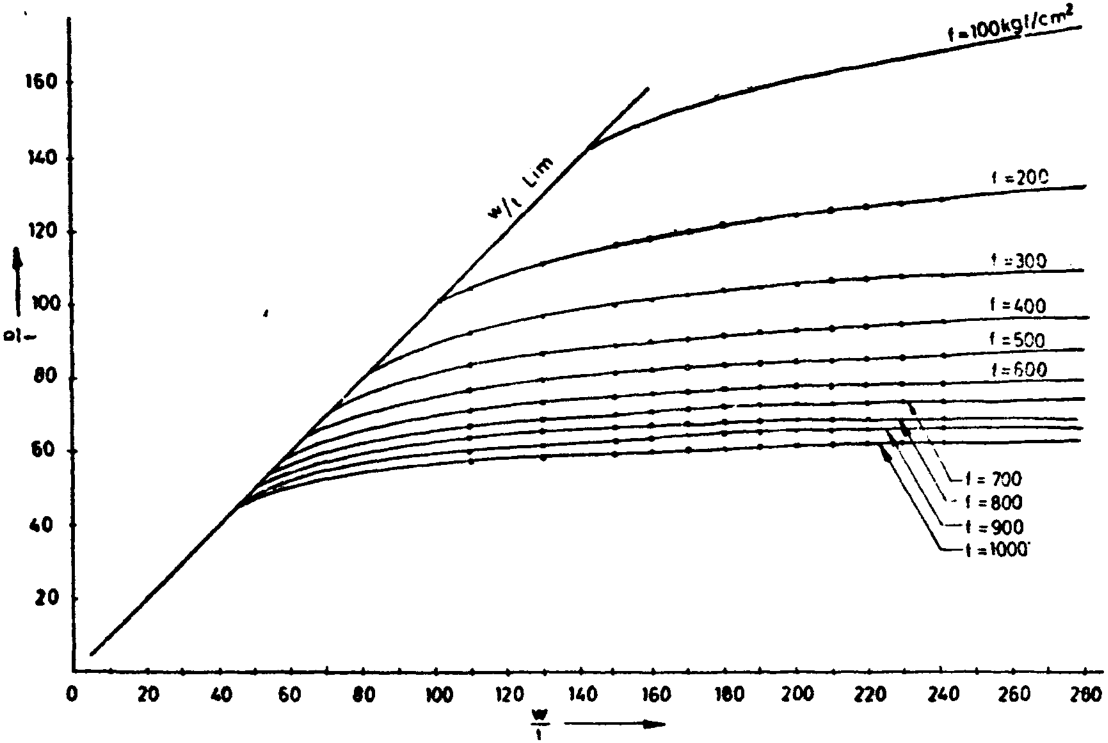


Fig. 25A $\frac{b}{t}$ Ratio for Load Determination (Non-tubular Sections)

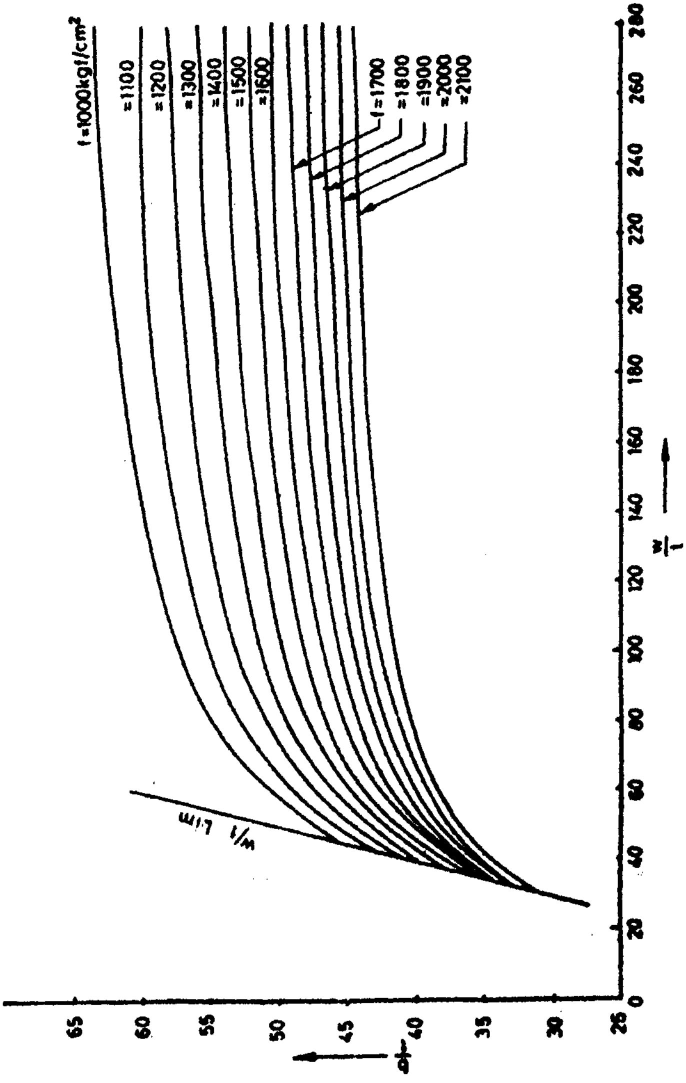


FIG. 25B $\frac{b}{t}$ RATIO FOR LOAD DETERMINATION (NON-TUBULAR SECTIONS)

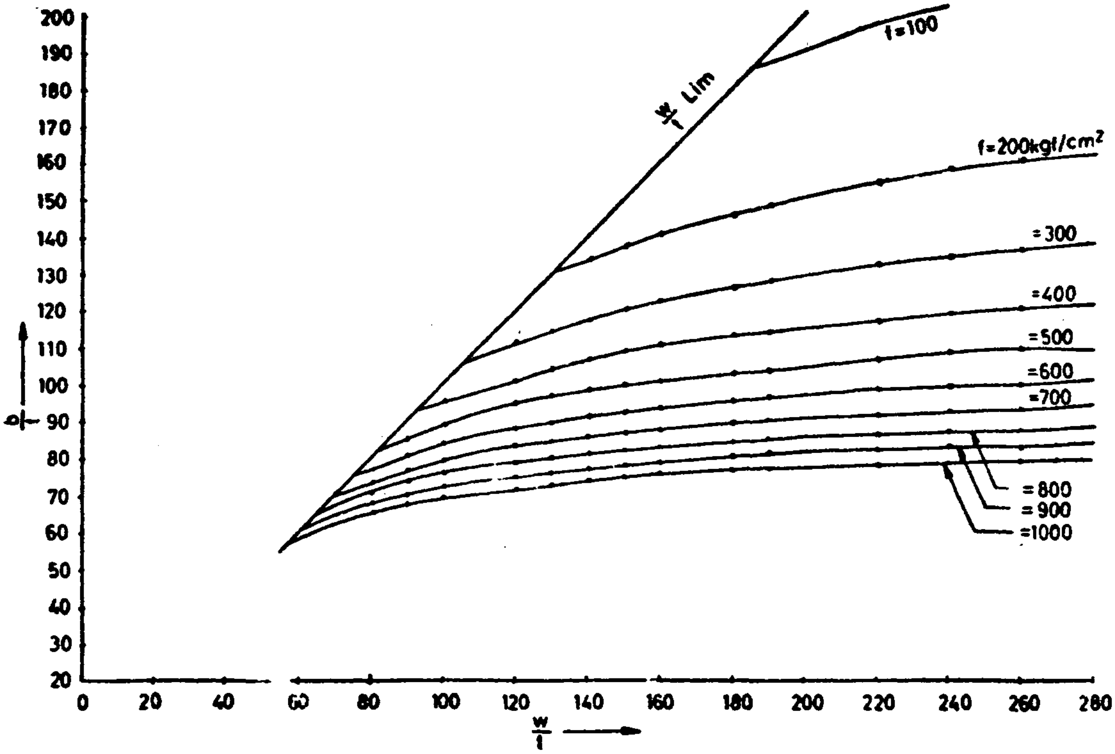


FIG. 26A $\frac{b}{t}$ RATIO FOR DEFLECTION DETERMINATION (NON-TUBULAR SECTIONS)

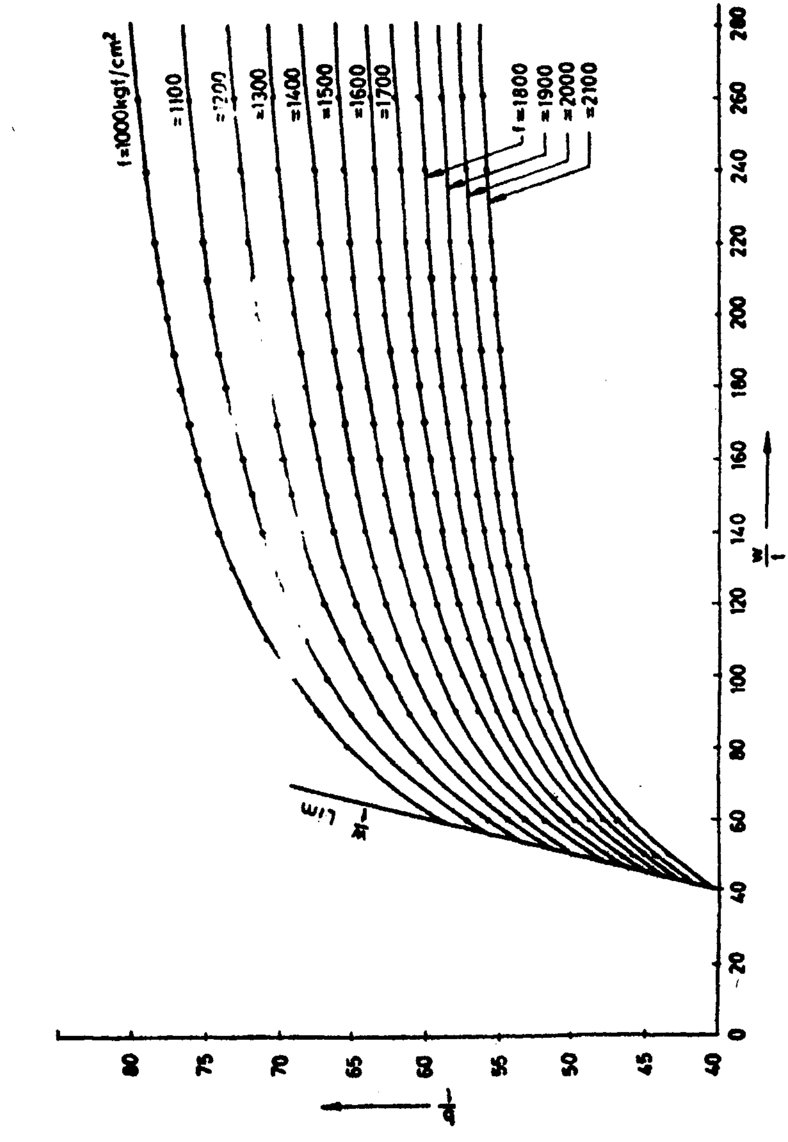


FIG. 26B $\frac{b}{t}$ RATIO FOR DEFLECTION DETERMINATION (NON-TUBULAR SECTIONS)

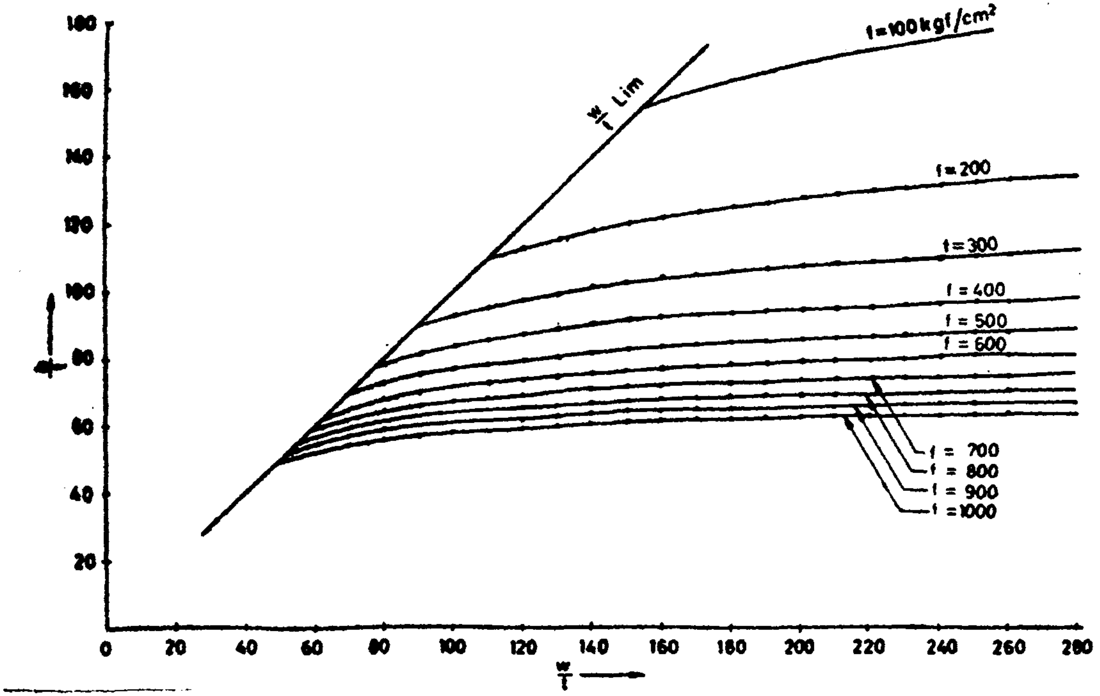


FIG. 27A $\frac{b}{t}$ RATIO FOR LOAD DETERMINATION (TUBULAR SECTIONS)

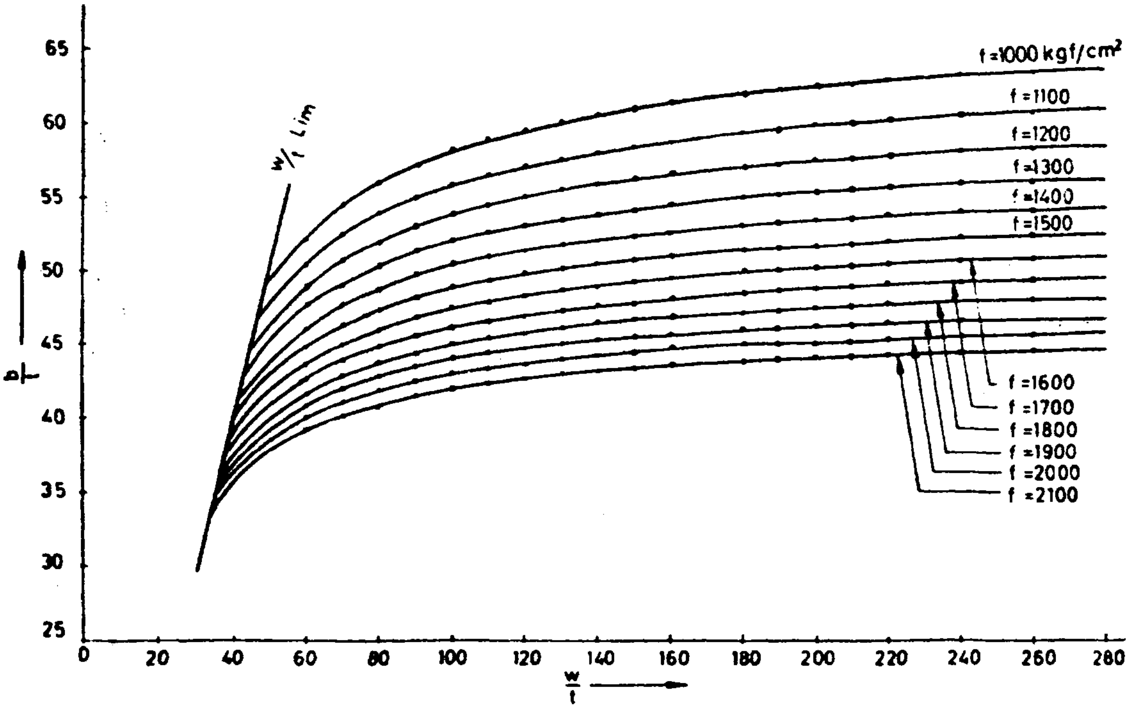


FIG. 27B $\frac{b}{t}$ RATIO FOR LOAD DETERMINATION (TUBULAR SECTIONS)

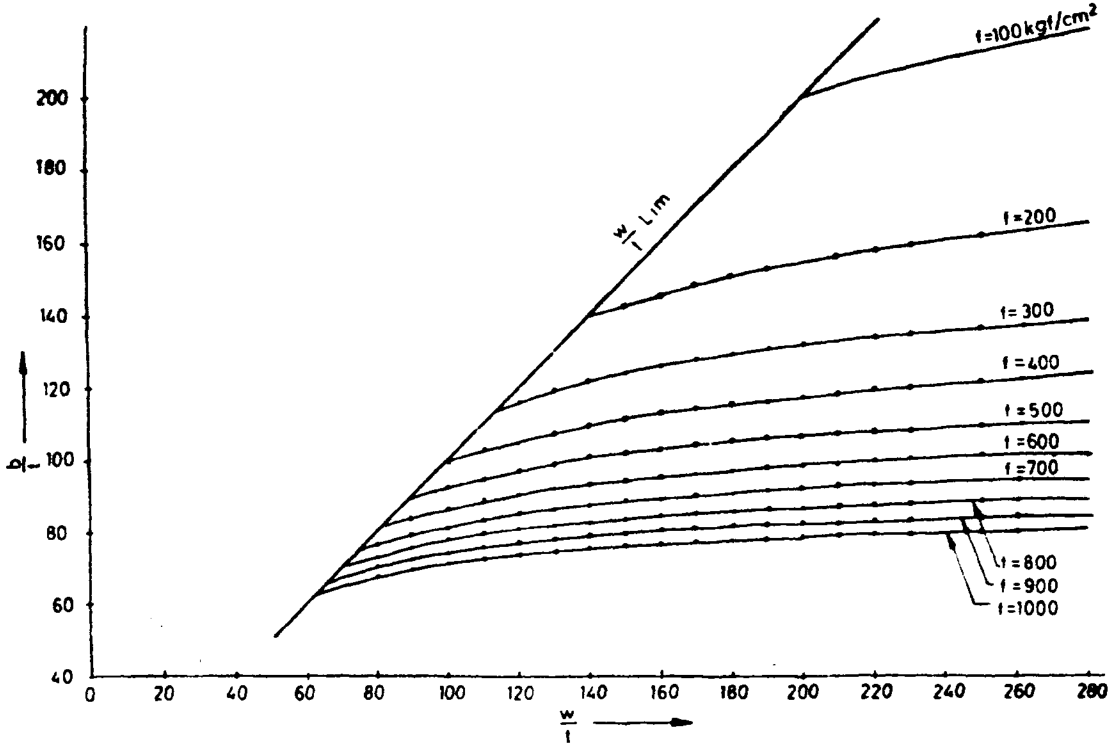


FIG. 28A $\frac{b}{t}$ – RATIO FOR DEFLECTION DETERMINATION (TUBULAR SECTIONS)

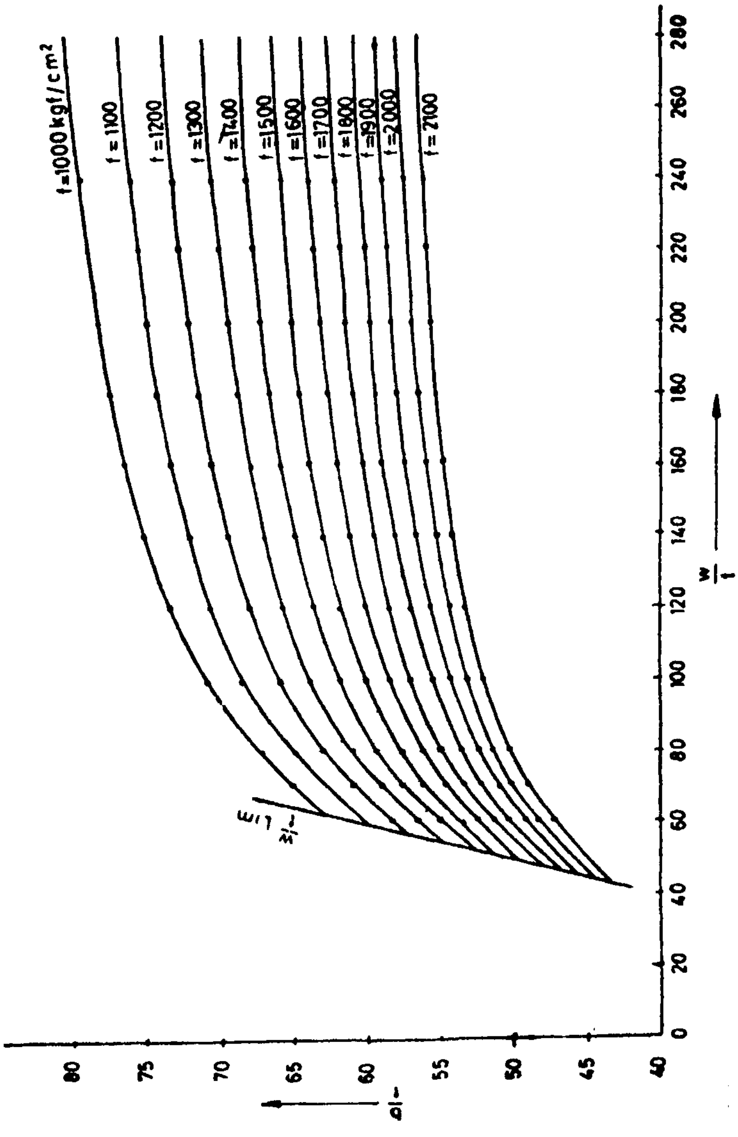


FIG. 28B $\frac{b}{t}$ RATIO FOR DEFLECTION DETERMINATION (TUBULAR SECTIONS)

4. STIFFENERS FOR COMPRESSION ELEMENTS

4.1 Clause **5.2.2.1** of IS : 801-1975 stipulates the minimum moment of inertia of a stiffened compression element by the relation:

$$I_{min} = 1{\cdot}83\, t^4 \sqrt{(w/t)^2 - 281\,200/F_y}$$

Values of (I_{min}/t^4) for different values of (w/t) and F_y have been tabulated in Table 3.

TABLE 3 MINIMUM MOMENT OF INERTIA OF EDGE STIFFENER (I_{min}/t^4)

$\frac{w}{t}$	YIELD POINT OF STEEL IN kgf/cm²			
	2 100	2 400	3 000	3 600
9·0	—	—	—	9·2
10·0	—	—	9·2	9·3
12·0	9·2	9·6	13·6	14·8
14·0	14·5	16·1	18·9	19·8
16·0	20·3	21·4	23·7	24·4
18·0	25·3	26·3	28·1	28·7
20·0	29 9	30·8	32·3	32·8
25·0	40·6	41·2	42·4	42·8
30·0	50·7	51·2	52·1	52·4
40·0	70·1	70·5	71·1	71·3
50·0	87·0	89·3	89·9	90·0
60·0	107·7	108·0	108·4	108·6
80·0	144·9	145·0	145·4	145·5
90·0	163·3	163·5	163·8	163·9

4.2 Clause **5.2.2.1** of IS : 801-1975 also stipulates the minimum overall depth d_{min} for stiffeners consisting of a simple line bent at right angles to the stiffened element by the relation:

$$d_{min} = 2{\cdot}8t \sqrt[6]{(w/t)^2 - 281\,200/F_y}$$

Values of $\frac{d_{min}}{t}$ have been tabulated for different values of (w/t) and F_y in Table 4.

5. COMPRESSION ON UNSTIFFENED ELEMENTS

5.1 Values of (w/t)$_{lim}$ for unstiffened compression elements for different values of F_y, calculated in accordance with the formulae contained in **6.2** of IS : 801-1975 are tabulated in Table 5.

TABLE 4 MINIMUM DEPTH OF SIMPLE LIP EDGE STIFFENERS (d_{min}/t)

(*Clause* 4.2)

$\frac{w}{t}$	YIELD STRESS OF STEEL IN kgf/cm²			
	2 100	2 400	3 000	3 600
9·0	—	—	—	—
10·0	—	—	4·8	4·8
12·0	4·8	4·9	5·4	5·6
14·0	5·6	5·8	6·0	6·2
16·0	6·2	6·3	6·5	6·6
18·0	6·7	6·8	6·9	7·0
20·0	7·1	7·2	7·3	7·3
25·0	7·9	7·9	8·0	8·0
30·0	8·5	8·5	8·5	8·6
40·0	9·4	9·5	9·5	9·5
50·0	10·2	10·2	10·3	10·3
60·0	10·9	10·9	10·9	10·9

TABLE 5 COMPRESSION ON UNSTIFFENED ELEMENTS

(*Clause* 5.1)

F_y kgf/cm²	LIMITING VALUES OF w/t FOR UNSTIFFENED COMPRESSION	
	$\frac{530}{\sqrt{F_y}}$	$\frac{1\,210}{\sqrt{F_y}}$
2 100	11·56	26·40
2 400	10·82	24·70
3 000	9·67	22·09
3 600	8·83	20·16

5.2 Allowable compressive stresses F_0 determined for different values of w/t as stipulated in **6.2**(a), (b), (c) and (d) of IS : 801-1975 have been given in Fig. 29 for F_y = 2 100, 2 400, 3 000 and 3 600 kgf/cm².

6. LATERALLY UNBRACED BEAMS

6.1 Clause **6.3**(a) of IS : 801-1975 stipulates the maximum allowable stress F_b on extreme fibres of laterally unsupported straight flexural members when bending is about the centroidal axis perpendicular to the web for either I-shaped section symmetrical about an axis in the plane of web or symmetrical channel-shaped sections by the relationship:

$$F_b = \frac{2}{3} F_y - \frac{F_y^2}{5{\cdot}4\pi^2 EC_b} \left(\frac{L^2 S_{xo}}{dI_{yo}} \right) \text{ when}$$

$\dfrac{L^2 S_{xo}}{dI_{yo}}$ is greater than $\dfrac{0{\cdot}36\pi^2 EC_b}{F_y}$ but less than $\dfrac{1{\cdot}8\pi^2 EC_b}{F_y}$

and $F_b = 0{\cdot}6\pi^2 \; EC_b \dfrac{dI_{yo}}{L^2 S_{xo}}$ if

$\dfrac{L^2 S_{xo}}{dI_{yo}}$ is equal to or greater than $\dfrac{1{\cdot}8\pi^2 EC_b}{F_y}$

The values of allowable bending stresses are given in Fig. 30 for various values of $\dfrac{L^2 S_{xo}}{dI_{yo}}$ and F_y.

6.2 Similarly, **6.3**(b) of IS : 801-1975 stipulates the maximum allowable stresses for point-symmetrical Z-shaped sections bent about the centroidal axis perpendicular to the web by the relationship:

$$F_b = \frac{2}{3} F_y - \frac{F_y^2}{2{\cdot}7\pi^2 EC_b} \left(\frac{L^2 S_{xo}}{dI_{yo}} \right) \text{ when}$$

$\dfrac{L^2 S_{xo}}{dI_{yo}}$ is greater than $\dfrac{0{\cdot}18\pi^2 EC_b}{F_y}$ but less than $\dfrac{0{\cdot}9\pi^2 EC_b}{F_y}$

and $F_b = 0{\cdot}3\pi^2 EC_b \dfrac{dI_{yo}}{L^2 S_{xo}}$ if

$\dfrac{L^2 S_{xo}}{dI_{yo}}$ is equal to or greater than $\dfrac{0{\cdot}9\pi^2 EC_b}{F_y}$

Values of allowable bending stresses are given in Fig. 31 for different values of $\dfrac{L^2 S_{xo}}{dI_{yo}}$ and F_y.

6.3 The values of the coefficients $\dfrac{0{\cdot}36\pi^2 E}{F_y}$ and $\dfrac{1{\cdot}8\pi^2 E}{F_y}$ for I-sections and symmetrical sections as given in **6.1** above and $\dfrac{0{\cdot}18\pi^2 E}{F_y}$ and $\dfrac{0{\cdot}9\pi^2 E}{F_y}$ for Z-sections for laterally unbraced beams as given in **6.2** above have been tabulated in Table 6.

TABLE 6 VALUES OF COEFFICIENTS

(*Clause* 6.3)

F_y kgf/cm²	I-SECTIONS AND SYMMETRICAL CHANNELS		Z-SECTIONS	
	$\frac{0\cdot36\pi^2E}{F_y}$	$\frac{1\cdot8\pi^2E}{F_y}$	$\frac{0\cdot18\pi^2E}{F_y}$	$\frac{0\cdot9\pi^2E}{F_y}$
2 100	3 509	17 545	1 755	8 775
2 400	3 070	15 350	1 535	6 675
3 000	2 456	12 280	1 228	6 140
3 600	2 047	10 235	1 024	5 120

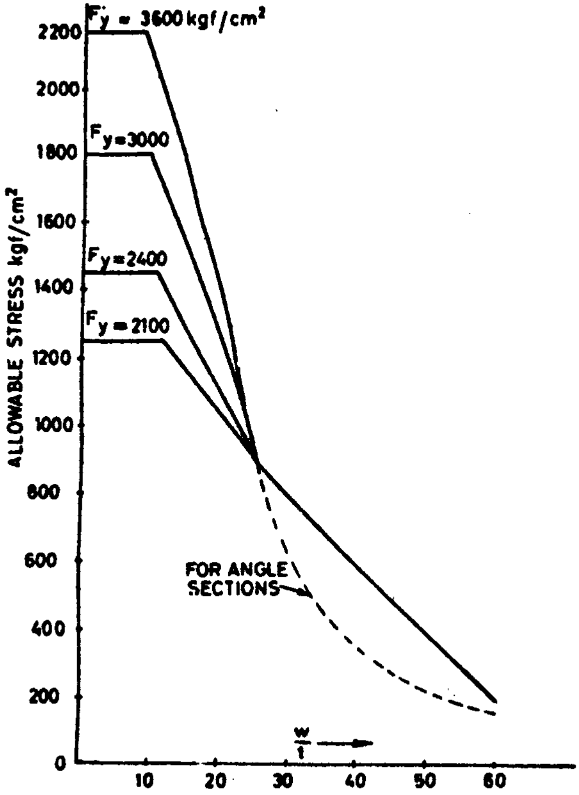


FIG. 29 ALLOWABLE COMPRESSIVE STRESS FOR UNSTIFFENED ELEMENTS, kgf/cm²

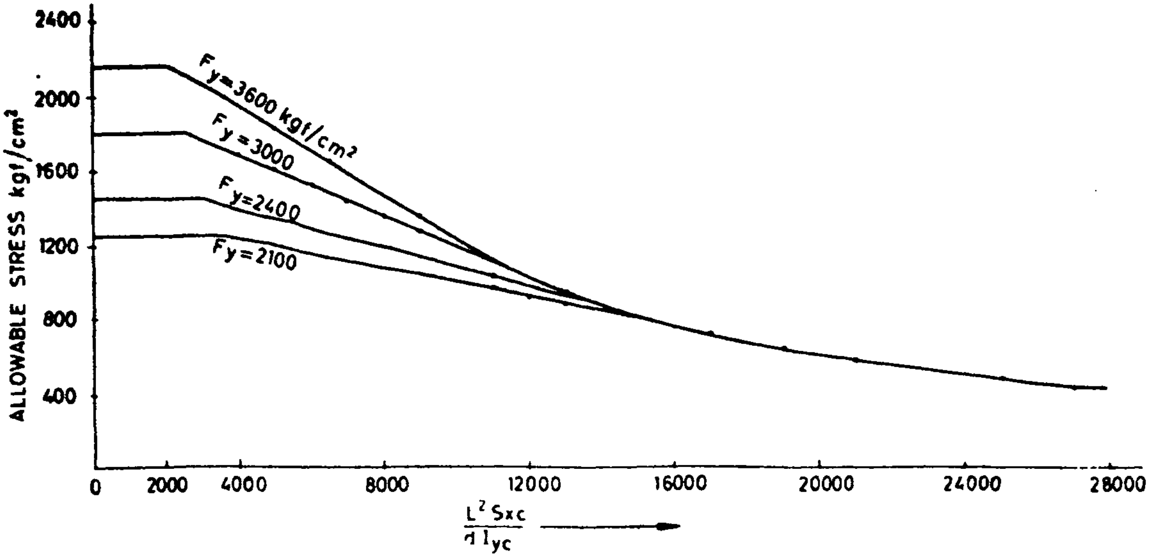


FIG. 30 ALLOWABLE BENDING STRESSES IN kgf/cm^2, FOR LATERALLY UNBRACED BEAMS (I-SECTIONS AND SYMMETRICAL CHANNELS)

6.4 In Fig. 30 and Fig. 31 bending coefficient C_b was assumed equal to 1. Values of C_b varies with different values of (M_1/M_2) by the relationship,

$C_b = 1{\cdot}75 + 1{\cdot}05\ (M_1/M_2) + 0{\cdot}3\ (M_1/M_2)^2$ subject to a minimum of 1 and maximum 2·3.

Table 7 gives values of the bending coefficient C_b for different values of (M_1/M_2) for laterally unbraced beams.

TABLE 7 BENDING COEFFICIENT C_b

$\frac{M_1}{M_2}$	C_b
−1·0	1·00
−0·8	1·10
−0·6	1·23
−0·4	1·38
−0·2	1·55
0·0	1·75
0·2	1·97
0·4	2·22
0·6	2·30
0·8	2·30
1·0	2·30

7. SHEAR STRESSES IN WEBS OF BEAMS

7.1 Clause **6.4.1** of IS : 801-1975 stipulates the maximum average shear stress F_v, on the gross area of a flat web by the relationship:

$F_v = \dfrac{1\,275\sqrt{F_y}}{h/t}$ with a maximum of 0·4 F_y when h/t not greater than $4\,590/\sqrt{F_y}$

and $F_v = \dfrac{5\,850\,000}{(h/t)^2}$ if h/t is greater than $4\,590/\sqrt{F_y}$

Based on the above formulae, Fig. 32 has been drawn giving allowable shear stresses for different values of h/t and F_y.

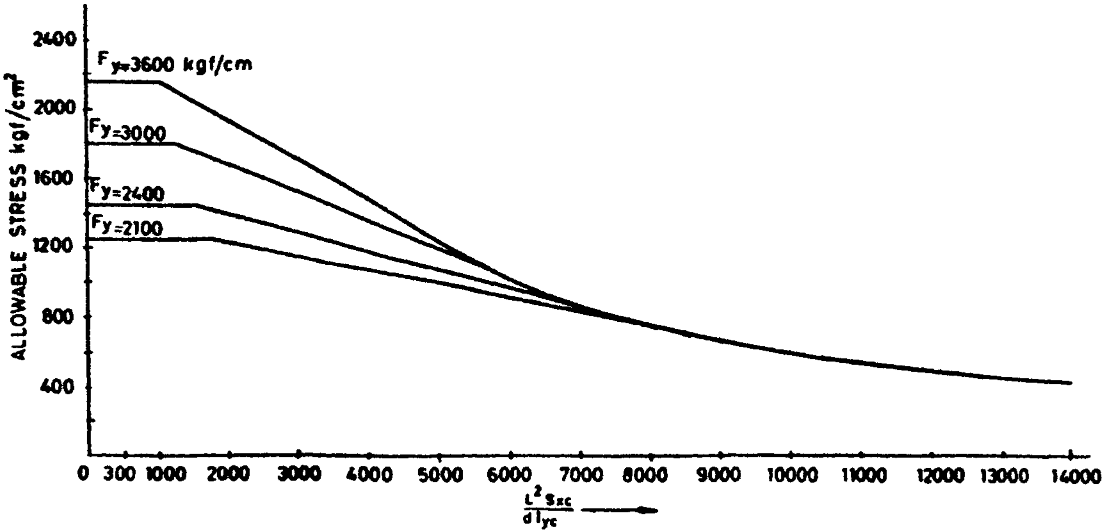


FIG. 31 ALLOWABLE BENDING STRESS IN kgf/cm², FOR LATERALLY UNBRACED BEAMS (Z-SECTIONS)

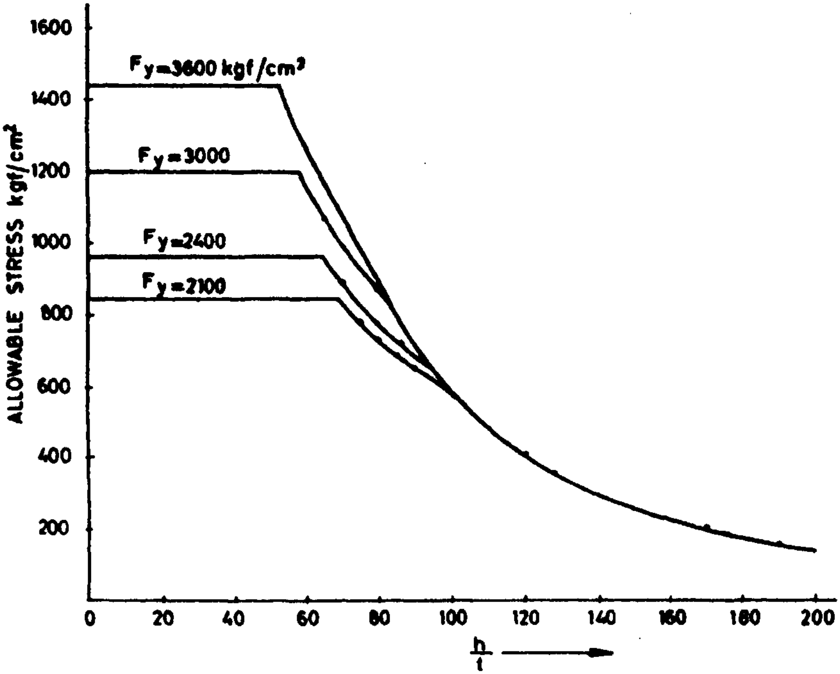


FIG. 32 ALLOWABLE SHEAR STRESS IN WEBS IN kgf/cm²

8. AXIALLY LOADED COMPRESSION MEMBERS

8.1 Compression members, not subjected to torsional-flexural buckling and braced against twisting, average axial stress P/A in compression is stipulated in 6.6.1.1(a) of IS : 801-1975 not to exceed as follows:

for KL/r less than $C_c/\sqrt{Q}$

$$F_{a1} = 12/23\,Q\,F_y - \frac{3\,(Q F_y)^2}{23\pi^2 E}\,(KL/r)^2$$

$$= 0{\cdot}522\;Q\;F_y - \left(\frac{Q F_y\,KL/r}{12\,500}\right)^2$$

and for KL/r equal to or greater than $C_c/\sqrt{Q}$

$$F_{a2} = \frac{12\pi^2 E}{23(KL/r)^2}$$

$$= \frac{10\,680\,000}{(KL/r)^2}$$

A graphical presentation giving allowable compressive stresses for steel of F_y = 2 100, 2 400, 3 000 and 3 600 kgf/cm^2 are given in Fig. 33, 34, 35 and 36 respectively for values of Q from 0·2 to 1·0.

8.2 Clause **6.6.1.1**(b) of IS : 801-1975 stipulates that when factor $Q = 1$, the steel is 2·29 mm or more in thickness and KL/r is less than C_c,

$$\text{allowable compressive stress, } F_{a1} = \frac{\left[1 - \dfrac{(KL/r)^2}{2C_c^2}\right] F_y}{\left[\dfrac{5}{3} + \dfrac{3}{8}\,\dfrac{KL/r}{C_c} - \dfrac{(KL/r)^3}{8\,C_c^3}\right]}$$

The values of F_{a1} as calculated from the formulae for different values of F_y are given in Fig. 37.

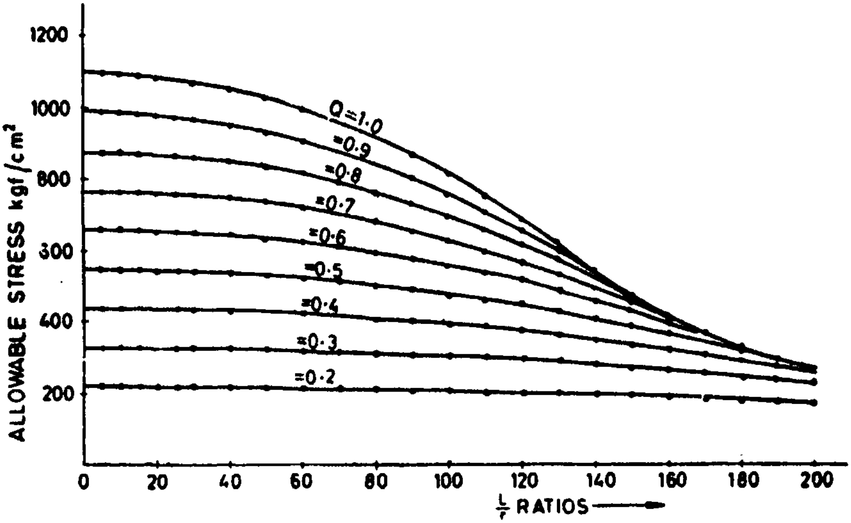


FIG. 33 ALLOWABLE COMPRESSIVE STRESS IN kgf/cm², FOR F_y = 2 100 kgf/cm²

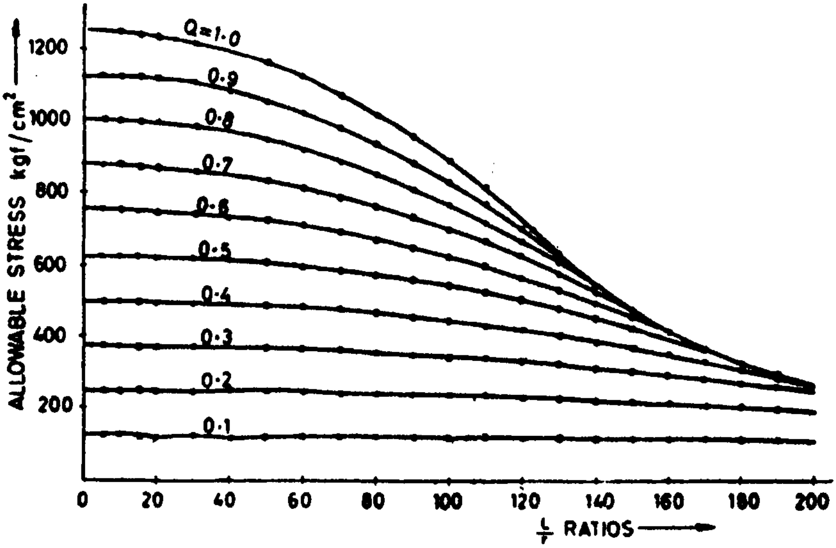


FIG. 34 ALLOWABLE COMPRESSIVE STRESS IN kgf/cm², FOR F_y = 2 400 kgf/cm²

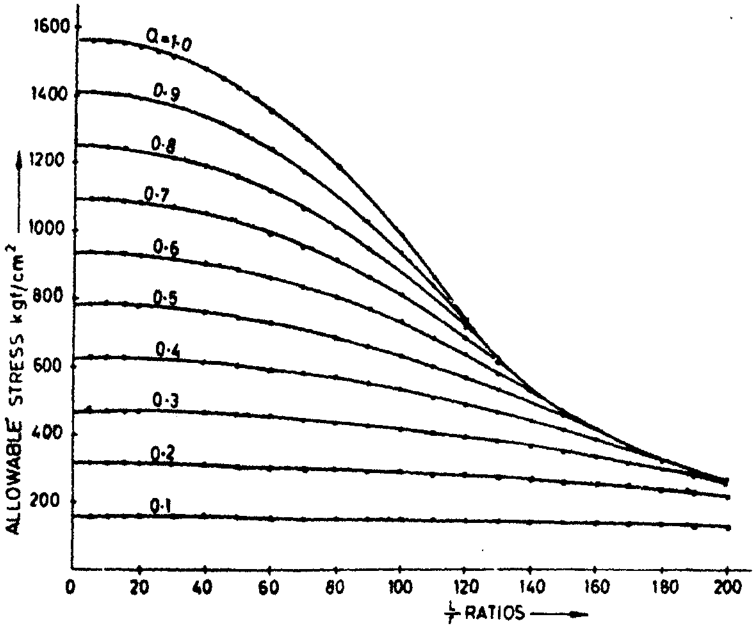


FIG. 35 ALLOWABLE COMPRESSIVE STRESS IN kgf/cm², FOR F_y = 3 000 kgf/cm²

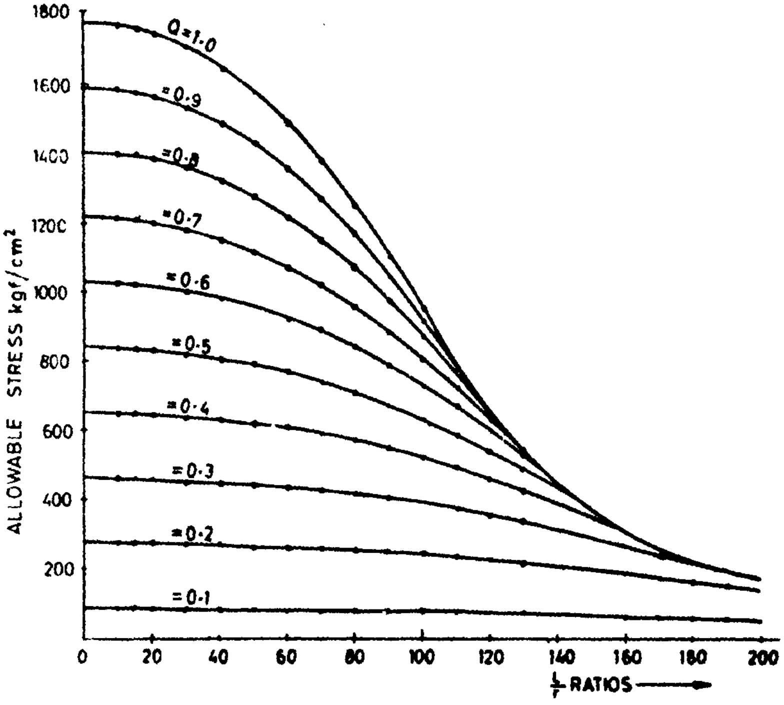


FIG 36 ALLOWABLE COMPRESSIVE STRESS IN kgf/cm², FOR F_y = 3 600 kgf/cm²

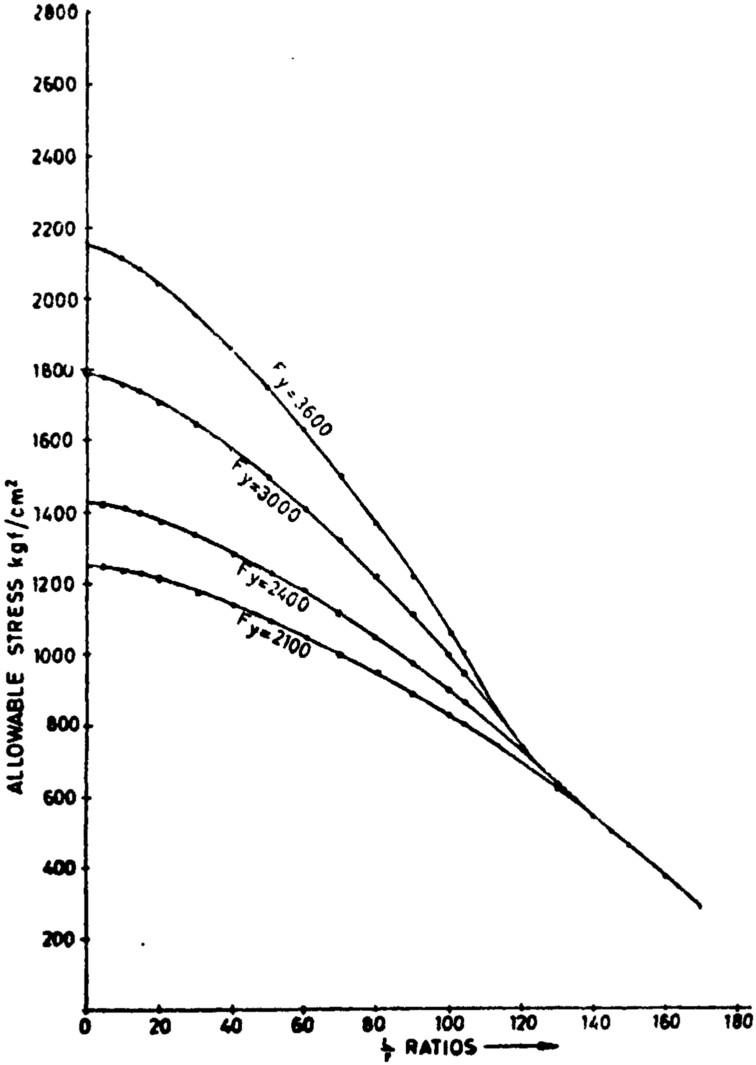


FIG. 37 ALLOWABLE COMPRESSIVE STRESS IN kgf/cm²

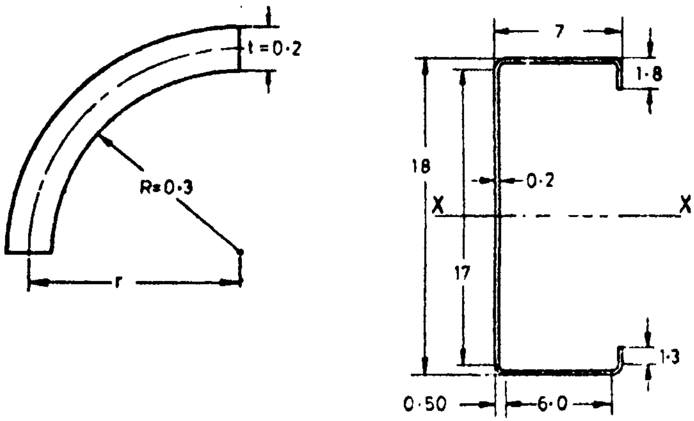
SECTION 3 DESIGN EXAMPLES

1. SCOPE

1.1 This section illustrates the application of the various code provisions of IS : 801-1975 along with figures and tables contained in Section 2 of the handbook in the design of various cold-formed steel structural members.

DESIGN EXAMPLE NO. 1 SECTIONAL PROPERTIES

To find the sectional properties by linear method of the section shown in Fig. 38.



$r = 0{\cdot}3 + 0{\cdot}2/2 = 0{\cdot}4$ cm

All dimensions in centimetres.

FIG. 38

Arc length for 90° corner = 1·57 × r = 0·628 cm

Distance of the C. G. from centre of arc = 0·637 × r

= 0·254 8 cm

Sl No.	*Elements*	*Dimension* cm	*Length* cm	*Distance from XX* d cm	Ad^2	*MI About Its Own C.G.*
1.	Web		17·0	0	—	409·42
2.	Lips	1·3 × 2	2·6	7·85	160·22	0·360
3.	Corners	0·628 × 4	2·51	8·75	192·17	
4.	Flanges	6·0 × 2	12·0	8·9	950·520	
	Total		34·11		1 302·91 = 1 712·69	409·780

(Continued)

DESIGN EXAMPLE NO. 1 SECTIONAL PROPERTIES – *Contd*

Area = 0·2 × 34·11 = 6·822 cm^2

MI = 0·2 × 1 712·69 = 342·54 cm^4

$$S_x = \frac{342{\cdot}54}{9{\cdot}0} = 38{\cdot}06$$

$$\text{Radius of gyration} = \sqrt{\frac{1\,712{\cdot}69}{34{\cdot}11}} = 7{\cdot}09 \text{ cm}$$

Neglecting the curves the quantities are as below:

Length = 17·8 + 6·8 + 1·7 + 1·7 = 34·8

Area = 34·8 × 0·2 = 6·96 cm^2

Error = 2·05 percent

$$MI = \frac{17{\cdot}8^3}{12} + 6{\cdot}8 \times 2\,(8{\cdot}9)^2 + 2 \times 1{\cdot}7 \times (8{\cdot}05)^2 + 2 \times \frac{1{\cdot}7^3}{12}$$

$$= 469{\cdot}98 + 1\,077{\cdot}25 + 220{\cdot}33 + 0{\cdot}82$$

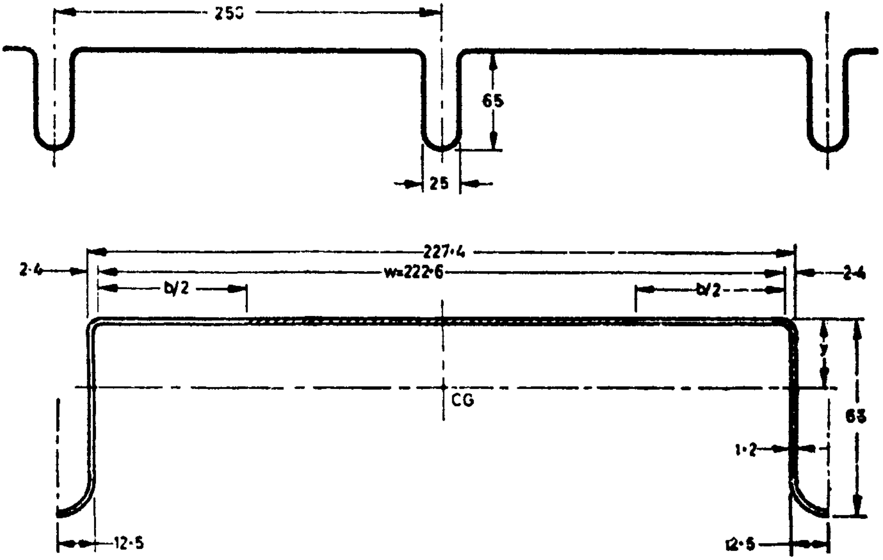
$$= 1\,768{\cdot}38 \text{ cm}^3$$

$$= 1\,768{\cdot}38 \times 0{\cdot}2 = 353{\cdot}676$$

Error = 3·24 percent

DESIGN EXAMPLE NO. 2 INTERMEDIATE SPAN ROOF DECK

The profile illustrated in Fig. 39 forms a roof deck. The gap in the top plate is limited to 25 mm to enable fibre boards and similar roof insulation to be used without danger of piercing. It is required to determine: (a) the resisting moment of this section as governed by the bending stress, and (b) the moment of inertia of the profile for deflection computation.



All dimensions in centimetres.

Fig. 39

a) *Allowable resisting moment*

First approximation

A compressive bending stress of 730 kgf/cm^2 is assumed. To simplify computation, the rounded corners may be assumed to be replaced by square corners as given in Fig. 40.

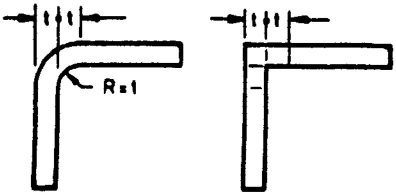


Fig. 40

(*Continued*)

The effective design width is determined in accordance with **5.2.1.1** of IS : 801-1975.

$$\frac{w}{t} = \frac{17{\cdot}26}{0{\cdot}12} = 143{\cdot}83$$

$$\left(\frac{w}{t}\right)_{\text{lim}} = \frac{1\,435}{\sqrt{\quad}} = \frac{1\,435}{\sqrt{730}} = 53{\cdot}1$$

$$\therefore \frac{b}{t} = \frac{2\,120}{\sqrt{730}}\left(1 - \frac{465}{(w/t)\sqrt{730}}\right) = 69$$

(b/t from Fig. 25A in Section 2 of the Handbook = 69)

$\therefore b = 69 \times 0{\cdot}12 = 8{\cdot}28$ cm.

Total effective width $b = 8{\cdot}28 + 0{\cdot}12 \times 2 = 8{\cdot}52$ cm

Area of elements $= 8{\cdot}52 \times 0{\cdot}12 + 6{\cdot}5 \times 0{\cdot}12 \times 2 + 1{\cdot}13 \times 0{\cdot}12 \times 2$

$= 1{\cdot}022 + 1{\cdot}56 + 0{\cdot}271$

$= 2{\cdot}853$ cm^2

Moment of the area about the top fibre

$= 1{\cdot}022 \times 0{\cdot}06 + 1{\cdot}56 \times 3{\cdot}25 + 0{\cdot}271 \times 6{\cdot}4 = 6{\cdot}865$ cm^3

$$\therefore y = \frac{6{\cdot}865}{2{\cdot}853} = 2{\cdot}406$$

$$\text{and } f_c = \frac{2{\cdot}406 \times 1\,250}{6{\cdot}5 - 2{\cdot}406} = 734{\cdot}6 \text{ kgf/cm}^2$$

Since the assumed stress, namely 730 kgf/cm^2, and the actual stress are more or less equal, the section properties can be calculated for the section.

Moment of inertia of the profile

$= 2 \times 0{\cdot}12 \times 6{\cdot}5^3/12 + 1{\cdot}56 \times 0{\cdot}844^2 + 0{\cdot}271\,(3{\cdot}974)^2 + 1{\cdot}024\,(2{\cdot}286)^2$

$= 15{\cdot}96$ cm^4

Section modulus $= 15{\cdot}96/4{\cdot}094 = 3{\cdot}967$ cm^3

Resisting moment of the section $= 3{\cdot}967 \times 1\,250$

$= 4\,958$ kgf. cm

b) *Moment of inertia for deflection calculation*

Actual sectional properties at design load are always larger than those computed for load determination. Correspondingly the actual top fibre stress is less than that computed for load determination and will be assumed as 600 kgf/cm^2.

$(w/t)_{\text{lim}} = 1\,850/\sqrt{600} = 75{\cdot}52$

Actual $w/t = 143{\cdot}83$

$\therefore$ b/t from Fig. 26A in Section 2 of the Handbook = 92 and

$b = 92 \times 0{\cdot}12 = 11{\cdot}04$ cm

(*Continued*)

DESIGN EXAMPLE NO. 2 INTERMEDIATE SPAN ROOF DECK – *Contd*

Total effective width = 11·04 + 0·24 = 11·28 cm

Area of the profile = 11·28 × 0·12 + 6·5 × 0·12 × 2 + 1·13 × 0·12 × 2
= 1·354 + 1·56 + 0·271 = 3·185 cm^2

Moment of area about top fibre = 1·354 × 0·06 + 1·56 × 3·25 + 0·271 × 6·4
= 0·081 + 5·07 + 1·734 = 6·885 cm^3

$\therefore y$ = 6·885/3·185 = 2·161 cm.

$$f_c = \frac{2\cdot161 \times 1\,250}{(6\cdot5 - 2\cdot161)} = 622\cdot5 \text{ kgf/cm}^2$$

This checks with the assumed values with sufficient accuracy.

Moment of area of the profile

$= 2 \times 0\cdot12 \times 6\cdot5^3/12 + 1\cdot56 \times 1\cdot089^2 + 0\cdot271 \times (4\cdot219)^2 + 1\cdot354 \times 2\cdot041^2$

$= 17\cdot807 \text{ cm}^4$

Check for flange curling

Fibre stress at design load as computed above = 622·7 kgf/cm^2

Average stress f_{av} = 622·7 × 11·28/17·74 = 395·94 kgf/cm^2

w_{max} = 17·26/2 = 8·63 cm.

w_{max} from **5.2.3**(d) of IS : 801-1975

$$w_{max} = \sqrt{\frac{19\,650\ t.d.}{f_{av}}} \times \sqrt[4]{\frac{100\ C_f}{d}}$$

$$\therefore C_f = \frac{w_{max}^4 \ . \ f^2_{av}}{(19\cdot65)^2 \times 10^8 \times t^2 d} = \frac{8\cdot63^4 \times 395\cdot94^2}{19\cdot65^2 \times 10^8 \times 0\cdot12^2 \times 6\cdot5}$$

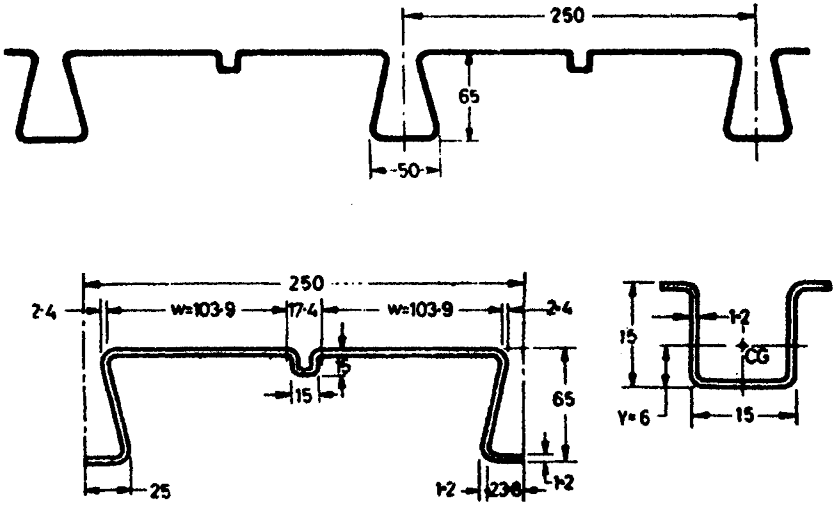
$= 0\cdot240$ cm.

This is less than 5 percent of the depth of the sections.

DESIGN EXAMPLE NO. 3 IMPROVED ROOF DECK

To determine the carrying capacity and other pertinent properties of the profile shown in Fig. 41.

This design example illustrates the manner in which the structural development work, more economical shapes are produced in successive designs. In Design Example 2, out of a total flat width of 143·8 t only 75 t is structurally effective. Hence, it is evident that the metal is not used economically. It is apparent that the top flange efficiency may be improved by providing an intermediate stiffener midway between the ribs. However, as a result of this improvement in the efficiency of compressive flange, the neutral axis would be located even closer to that flange than in Design Example 2. In general, the best location for the neutral axis is as close to the mid-depth as possible so as to minimize the amount of unstressed material. To compensate for the improved top flange efficiency, it is desirable to improve the balance of material above and below the axis by increasing the width of the rib, and thereby of the bottom flange. It is also necessary to prevent damage to the roof insulation material that the gap in the top plate should not exceed 25 mm. To do this and to add material to the bottom flange wedge-shaped ribs have been provided.



All dimensions in millimetres.

FIG. 41

(*Continued*)

DESIGN EXAMPLE NO. 3 IMPROVED ROOF DECK — *Contd*

a) *Allowable Resisting Moment*

Actual Moment of Inertia of the Stiffener

Area of elements =

$2 \times 0·12 \times 0·12 = 0·028\ 8$

$2 \times 1·5 \times 0·12 = 0·360\ 0$

$1 \times 1·26 \times 0·12 = \underline{0·151\ 2}$

$0·540\ 0\ \text{cm}^2$

Moment of the area about bottom flange

$= 0·028\ 8 \times 1·44 + 0·36 \times 0·75 + 0·151 \times 0·06$

$= 0·041\ 5 + 0·27 + 0·009\ 072 = 0·320\ 572\ \text{cm}^3$

$\therefore y = 0·320\ 57/0·54 = 0·6\ \text{cm}$

Moment of inertia $= 0·028\ 8 \times (0·84)^2 + 2 \times 0·12 \times 1·5^2/12 + 0·36 \times (0·15)^2 + 0·151\ 2 \times (0·54)^2$

$= 0·020\ 3 + 0·067\ 5 + 0·008\ 1 + 0·004\ 415$

$= 0·074\ 005\ \text{cm}^4$

The central portion, with all necessary dimensions, is shown above and for simplicity properties are computed only for this central portion. For such sections, it is first necessary to design the stiffener to have the required rigidity. Subsequently, the sectional properties are computed in a manner similar to that of Example 2.

$w/t = 103·9/1·2 = 86·58$

Assume a stress of 590 kgf/cm^2 at the top flange. Effective width ratio b/t for $f = 590$ kgf/cm^2 and $w/t = 86·58$ is 68·0 (from Fig. 25A in Section 2 of the handbook).

$b'/t = b/t - 0·10\ (w/t - 60)$

$= 68 - 0·1\ (86·58 - 60) = 65·41$

$b' = 65·41 \times 0·12 = 7·85$

Effective area of stiffener $= \alpha.A_{eff}$

The value of α from Table 2 in Section 2 for $w/t = 86·5$ and $b/t = 68$ is 0·82.

Effective area of stiffener $= 0·82 \times 0·54 = 0·442\ 8$

Effective width of top flange excluding stiffener $= 2\ (7·85 + 0·12)$
$= 15·94$ cm

(*Continued*)

DESIGN EXAMPLE NO. 3 IMPROVED ROOF DECK — *Contd*

Element (1)	*Area, A* (2) cm^2	*Distance from Top Fibre, y* (3) cm	*A.y* (4) cm^3
Top flange	15·94 × 0·12 = 1·912 8	0·06	0·114 7
Stiffener	0·443	0·90	0·398 7
Webs	2 × 6·5 × 0·12 = 1·56	3·25	5·070
Bottom flange	2 × 2·38 × 0·12 = 0·571 2	6·49	3·68
	4·487 cm^2		9·263 4 cm^3

$y = 9{\cdot}263\,4/4{\cdot}487 = 2{\cdot}064\,5$ cm

$f_c = 1\,250 \times 2{\cdot}064\,5/(6{\cdot}5 - 2{\cdot}064\,5) = 581{\cdot}8$ kgf/cm^2

This is quite near to the assumed value and the section properties can be determined for these values.

$$I_{xx} = 1{\cdot}912\,8 \times 2{\cdot}00^2 + 0{\cdot}443\,(1{\cdot}064)^2 + 2 \times 0{\cdot}12 \times 6{\cdot}5^3/12 + 1{\cdot}56 \times (1{\cdot}19)^2 + 0{\cdot}571\,2\,(4{\cdot}43)^2 = 22{\cdot}416 \text{ cm}^4$$

Section modulus = 22·416/4·435 5 = 5·054 cm^3

∴ Allowable resisting moment = 1 250 × 5·054 = 6 317·5 kgf. cm

b) *Moment of Inertia I for Deflection Calculation*

$w/t = 86{\cdot}5$

Assume a stress of 520 kgf/cm^2

b/t for deflection calculation is 80·5 from Fig. 26A in Section 2

$b'/t = 80{\cdot}5 - 0{\cdot}1\,(86{\cdot}5 - 60) = 77{\cdot}85$

$b' = 77{\cdot}85 \times 0{\cdot}12 = 9{\cdot}34$ cm

For effective area of stiffener, α from Table 2 in Section 2 is 0·94.

Effective area of stiffener = 0·94 × 0·54 = 0·507 cm^2

Total width of flange 2 (9·34 + 0·12) = 18·92 cm

(*Continued*)

DESIGN EXAMPLE NO. 3 IMPROVED ROOF DECK — *Contd*

Cross-Sectional Properties

Element	*Area, A*	*Distance from Top Fibre, y*	*A.y*
	cm^2	cm	cm^3
Top flange	18·92 × 0·12 = 2·270	0·06	0·136
Stiffener	0·507	0·9	0·456
Webs	2 × 6·5 × 0·12 = 1·56	3·25	5·071
Bottom flange	2 × 2·38 × 0·12 = 0·571	6·44	3·660
	4·908 cm^2		9·322 cm^3

$\therefore\ y = 9{\cdot}322/4{\cdot}908 = 1{\cdot}899$ cm, and

$$f_c = \frac{1\,250 \times 1{\cdot}899}{6{\cdot}5 - 1{\cdot}899} = 515{\cdot}92 \text{ kgf/cm}^2$$

This is near to the assumed value with sufficient accuracy

$$\therefore\ I = 2{\cdot}27 \times (1{\cdot}84)^2 + 0{\cdot}507 \times (1)^2 + 2 \times 0{\cdot}12 \times 6{\cdot}5^3/12 + 1{\cdot}56 \times (1{\cdot}35)^2 + 0{\cdot}571 \times (4{\cdot}54)^2$$

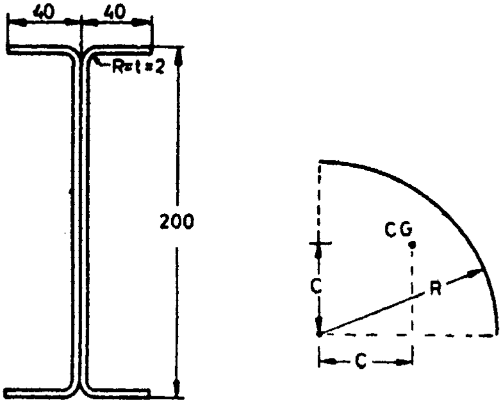
$$= 7{\cdot}685 + 0{\cdot}507 + 5{\cdot}49 + 2{\cdot}84 + 11{\cdot}767$$

$$= 28{\cdot}284 \text{ cm}^4$$

DESIGN EXAMPLE NO. 4A BEAM STRENGTH CALCULATION

A floor joist consists of two channels welded back to back to form an unstiffened I-Section. It carries a uniformly distributed load of 250 kgf/m over a span of 4 m. The limiting deflection is 1/325 of span.

It is required to determine if this section will meet the deflection limitations, to check the adequacy of the given section in bending for the span and loading mentioned above, and to determine the maximum allowable spacing of lateral braces.



All dimensions in millimetres.

FIG. 42

Maximum deflection = 400/325 = 1·23 cm

The moment of inertia of the section is determined by linear method as follows:

$R = t + t/2 = 2 + 1 = 3$ mm

$l = 1{\cdot}57\,R = 1{\cdot}57 \times 3 = 4{\cdot}71$ mm

Moment of inertia of corner $I_{0y} = 0{\cdot}149\,R^3$

$= 0\ 149 \times 3^3$

$= 4$ mm^3 negligible

Area $lt = 0{\cdot}471 \times 2 = 0{\cdot}942$ mm^2

$c = 0{\cdot}637\,R = 0{\cdot}637 \times 3{\cdot}0 = 1{\cdot}911$ mm

$$I_{xx} = 4 \times 0{\cdot}942\,(100 - 1{\cdot}9)^2 + 4 \times 36 \times 2 \times (100 - 2)^2 + 2 \times 2 \times 192^3/12$$

$$= 36\,300 + 2\,766\,000 + 2\,360\,000 = 5\,162\,300 \text{ mm}^4 = 516{\cdot}23 \text{ cm}^4$$

(Continued)

DESIGN EXAMPLE NO. 4A BEAM STRENGTH CALCULATION — *Contd*

The maximum deflection in a uniformly loaded beam that occurs at the mid-span is equal to

$$\frac{5}{384} \times \frac{wL^4}{EI} = \frac{5}{384} \times \frac{250 \times 4 \times 400^3}{2\,074\,000 \times 516{\cdot}23} = 0{\cdot}788 \text{ cm}$$

This is less than the permissible deflection of 1·23 cm, thus satisfying the deflection requirement. Section modulus of the beam $= Z_{xx} = \frac{516{\cdot}23}{10}$

$= 51{\cdot}62$ cm.

Flat width ratio of flange element = 36/2 = 18

Allowable compressive stress = 1 090 kgf/cm^2

(From Fig. 29 in Section 2)

Maximum bending moment at the centre of the beam = 250 × 4^2/8 × 100

= 50 000 kgf.cm.

Actual bending stress developed = 50 000/52·163 = 960 kgf/cm^2

This is less than the allowable stress, thus satisfying the beam strength requirement.

Total width of the sheet used = 2 (2 × 36 + 2 × 4·71 + 192)

= 2 × 273·4 = 546·8 mm

= 54·68 cm

Area of cross section = 54·68 × 0·2 = 10·936 cm^2

Weight per metre run = 10·936 × 0·785 = 8·48 kg

Bracing requirement (Referring **6.3** of IS : 801-1975)

$C_b = 1$

$$\frac{1{\cdot}8\pi^2 E C_b}{F_y} = \frac{1{\cdot}8 \times 9{\cdot}87 \times 2\,074\,000 \times 1}{2\,100} = 17\,546{\cdot}0$$

Assume $L = 100$ cm $\qquad \frac{L^2 S_{xc}}{dI_{yc}} = \frac{100^2 \times 52{\cdot}16}{20 \times 2{\cdot}515} = 10\,369{\cdot}8$

$$\frac{L^2 S_{xc}}{dI_{yc}} < \frac{1{\cdot}8\pi^2 E C_b}{F_y} \quad \therefore F_b = \left[\frac{2}{3} F_y - \frac{F_y{}^2}{5{\cdot}4\pi^2 E C_b} \times \left\{\frac{L^2 S_{xc}}{dI_{yc}}\right\}\right]$$

$$1\,090 = \left[\frac{2}{3} \times 2\,100 - \frac{2\,100^2}{5{\cdot}4 \times 9{\cdot}87 \times 2\,074\,000} \times \frac{52{\cdot}16\,L^2}{20 \times 2{\cdot}515}\right]$$

Solving for *L*:

$$L^2 = \frac{310 \times 9{\cdot}87 \times 5{\cdot}4 \times 2\,074\,000 \times 20 \times 2{\cdot}515}{2\,100 \times 2\,100 \times 52{\cdot}16}$$

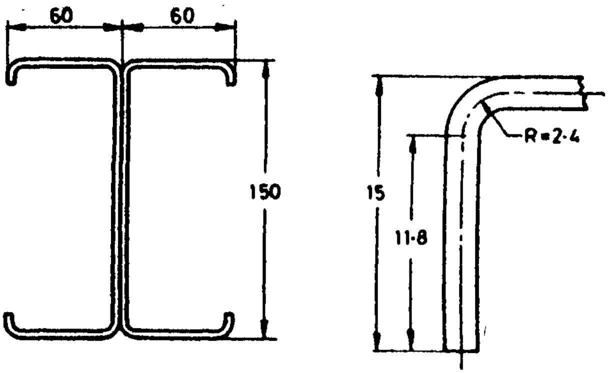
$= 7\,493{\cdot}3$

$L = 86{\cdot}56$ cm

$\frac{L^2 S_{xc}}{dI_{yc}}$ is still less than $1{\cdot}8\pi^2 E C_b/F_y$. Hence the allowable stress expression used is correct and the spacing of bracing comes as 86·56 cm.

DESIGN EXAMPLE NO. 4B BEAM STRENGTH CALCULATION

The data are the same as for Design Example 4A except that beam section consists of two channel welded back to back to form a stiffened I-section as shown.



DETAILS OF LIP

All dimensions in millimetres.

FIG. 43

Check the lip for minimum strength required for stiffener.

The depth of the stiffener (lip) and its moment of inertia are determined according to 5.2.2.1 of IS : 801-1975.

$w/t = 53{\cdot}6/1{\cdot}6 = 33{\cdot}5$

Minimum depth of the lip required

$= 2{\cdot}8\,t\,\sqrt[6]{(w/t)^2 - 281\,200/F_y}$ but not less than $4{\cdot}8t$

$= 2{\cdot}8 \times 1{\cdot}6\,\sqrt[6]{33{\cdot}5^2 - 281\,200/2\,100}$

$= 2{\cdot}8 \times 1{\cdot}6\,\sqrt[6]{1\,122{\cdot}25 - 133{\cdot}90}$

$= 14{\cdot}11$ mm

and $4{\cdot}8t = 4{\cdot}8 \times 1{\cdot}6 = 7{\cdot}68$ mm

Hence depth of lip provided = 15 mm is O. K.

Properties of the 90° corner

$R = 1{\cdot}6 + 0{\cdot}8 = 2{\cdot}4$ mm

$l = 1{\cdot}56 \times 2{\cdot}4 = 3{\cdot}77$ mm

$c = 0{\cdot}637 \times 2{\cdot}4 = 1{\cdot}53$ mm

Area $= 3{\cdot}77 \times 1{\cdot}6 = 5{\cdot}032$ mm^2

(Continued)

DESIGN EXAMPLE NO. 4B BEAM STRENGTH CALCULATION — *Contd*

I_{xx} of the entire section

Web = $2 \times 1{\cdot}6 \times 143{\cdot}6^3/12 = 788\ 000$

$$\text{Straight portion of lips} = \frac{4 \times 1{\cdot}6\,(\,11{\cdot}8^3 + 11{\cdot}8 \times 65{\cdot}9^2\,)}{12} = 328\ 876$$

Eight corners $8 \times 5{\cdot}03^2 \times 73{\cdot}32^2 = 216\ 000$

Flanges $= 4 \times 53{\cdot}16 \times 1{\cdot}6 \times 74{\cdot}2^2 = 1\ 875\ 000$

Total $= 3\ 207\ 876 = 320{\cdot}8\ \text{cm}^4$

If only part of the top flange is effective, the neutral axis is below mid-depth, and the compressive stress is larger than the tension stress and hence governs. If the gross section of the top flange is effective, top and bottom flange stresses are equal. Hence in either case the design stress in the top flange is known and is 1 250 kgf/cm². *w/t* limit for *f* of 1 250 kgf/cm² is 40·59 for load determination and 52·33 for deflection determination but the actual value is 33·5 which is less than both the limits and hence the full section is effective.

∴ Modulus of section = 320·8/7·5 = 42·77

Check for deflection

As the gross section is effective for deflection also, actual deflection = $5 \times 250 \times 4 \times 400^3/(\,384 \times 2\ 074\ 000 \times 320{\cdot}8\,) = 1{\cdot}253$ cm

Check for beam strength

The maximum bending moment in a uniformly loaded simple beams

$= 250 \times 4^2 \times 100/8 = 50\ 000$ kgf.cm

Bending stress = 50 000/42·77 = 1 169·04 kgf/cm²

This is less than the allowable stress of 1 250 kgf/cm². Total area of the section = 9·08 cm²

Weight per metre of the beam = 9·08 × 0·785 = 7·12 kg/m

Saving over the unstiffened section of design example is

$$\frac{8{\cdot}48 - 7{\cdot}12}{8{\cdot}48} \times 100 = 16{\cdot}05 \text{ percent}$$

Bracing requirement

$$\text{Linear } I_{yy} = 4 \times 1{\cdot}18 \times (\,5{\cdot}92\,)^2 + 4 \times 0{\cdot}377 \times 5{\cdot}833^2 + 4 \times \frac{5{\cdot}36^3}{12}$$
$$+ 4 \times 5{\cdot}36 \times 3^2 + 4 \times 0{\cdot}377\,(\,1{\cdot}67\,)^2 + 14{\cdot}36\,(\,0{\cdot}08\,)^2$$
$$= 466{\cdot}9\ \text{cm}^3$$

Actual $I_{yy} = 466{\cdot}9 \times 0{\cdot}16 = 74{\cdot}7\ \text{cm}^4$

$$I_{yo} = \frac{I_{yy}}{2} = 37{\cdot}35\ \text{cm}^4$$

(*Continued*)

DESIGN EXAMPLE NO. 4B BEAM STRENGTH CALCULATION — *Contd*

$$\frac{0{\cdot}36 \times \pi^2 \times E}{f_y} = \frac{0{\cdot}36 \times 9{\cdot}87 \times 2\,074\,000}{2\,100} = 3\,509{\cdot}2$$

Assume free bending length as 200 cm (that is a centre bracing)

Then $\frac{L^2 S_{xc}}{d I_{yc}} = \frac{200^2 \times 42{\cdot}47}{15 \times 37{\cdot}35} = 3\,032{\cdot}21$

This is less than $0{\cdot}36\pi^2 E/F_y$, therefore allowable stress is 1 250 itself. Hence provide a bracing at the middle of the beam.

To check the adequacy of connecting the two component channels by spot welds

According to **7.2.2** of IS : 801-1975 allowable shear force per spot of a sheet thickness 1·6 mm is 330 kgf.

According to **7.3** of IS : 801-1975

Spacing $S_{max} = 2.g.T_s/m.q \ngtr L/6$

The distance of shear centre is given by

$$m = \frac{w_f d.t}{4\,I_x}\left[w_f d + 2\,d_1 \left(d - 4\,d_1^2/3d \right) \right]$$

$$= \frac{5{\cdot}84 \times 15 \times 0{\cdot}16}{4 \times 160{\cdot}4}\left[5{\cdot}84 \times 15 + 2 \times 1{\cdot}5 \left(15 - 4 \times 1{\cdot}5^2/3 \times 15 \right) \right]$$

$$= \frac{5{\cdot}84 \times 15 \times 0{\cdot}16 \times 132}{4 \times 160{\cdot}4} = 2{\cdot}883 \text{ cm}$$

and $S_{max} = \frac{2 \times 10{\cdot}0 \times 330}{2{\cdot}883 \times 3 \times 250/100} = 305$ cm

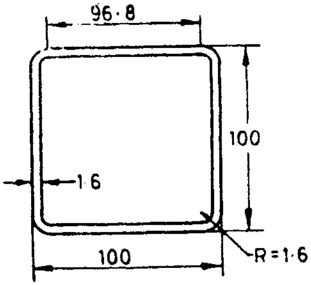
$L/6 = 400/6 = 66{\cdot}6$

Provide spacing of 60 cm centre-to-centre.

DESIGN EXAMPLE NO. 5 AXIALLY LOADED COMPRESSION MEMBER

To find the column section properties and allowable axial load for the column section. Length of the column = 2·75 m

Steel used F_y = 3 000 kgf/cm²



All dimensions in millimetres.

FIG. 44

Thickness of sheet = 1·6 mm

The properties of the 90° corner

$R = 1{\cdot}6 + 0{\cdot}8 = 2{\cdot}4$ mm

$L = 1{\cdot}57 \times 2{\cdot}4 = 3{\cdot}77$ mm

$c = 0{\cdot}637\,R = 0{\cdot}637 \times 2{\cdot}4 = 1{\cdot}53$ mm

Moment of inertia about centroidal axis

Element	*Length, L* mm	*y* mm	Ly^2 mm³	*Own Axis*
Flanges	2 × 96·8 = 193·6	49·2	468 635·9	0
Webs	2 × 96·8 = 193·6	0	—	151 173·2
Corners	4 × 3·77 = 15·08	49·93	37 594·5	
	402·28		506 230·4 + 151 173·2 = 657 403·6 mm³	

Moment of inertia 657 403·6 mm³ = 657·403 cm³

Actual moment of inertia = 657·403 × 0·16 = 105·18 cm⁴

(*Continued*)

DESIGN EXAMPLE NO. 5 AXIALLY LOADED COMPRESSION MEMBER — *Contd*

Area $= 40{\cdot}23 \times 0{\cdot}16 = 6{\cdot}428$ cm^2

Radius of gyration $= \sqrt{\dfrac{105{\cdot}18}{6{\cdot}428}} = 4\ 045$ cm

$w/t = 96{\cdot}8/1{\cdot}6 = 60{\cdot}5$

Effective width b/t for a stress of 1 250 kgf/cm^2 for tubular sections from Fig. 27B in Section 2 $= 48$, reduction in effective width $(60{\cdot}5 - 48)\ 0{\cdot}16 = 12{\cdot}5 \times 0{\cdot}16 = 2$ cm

Reduction in effective area $= 2 \times 0{\cdot}16 = 0{\cdot}32$

Total effective area $= 6{\cdot}428 - 4 \times 0{\cdot}32 = 5{\cdot}148$ cm^2

$$Q = \frac{A_{\text{eff}}}{A} = \frac{5{\cdot}148}{6{\cdot}428} = 0{\cdot}801$$

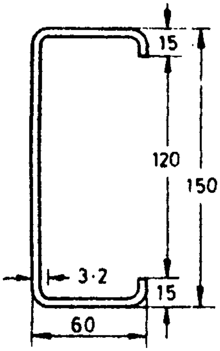
$l/r = 275/4{\cdot}045 = 67{\cdot}98$

Allowable stress from Fig. 35 in Section 2 $= 1\ 080$ kgf/cm^2

Allowable load $1\ 080 \times 6{\cdot}428 = 6\ 942$ kgf.

DESIGN EXAMPLE NO. 6 WALL STUD BRACED BY WALL SHEATHING-AXIAL COMPRESSION MEMBER

To find the allowable load P on the section shown in Fig. 45.



All dimensions in millimetres.

FIG. 45

Height of column = 4·5 m

Material of sheathing is standard density wood with Kw (modulus of elastic support) = 60 kg/cm

Wall sheathing of sufficient rigidity is attached to each of the flanges of the channel section, which prevents the channel section from buckling in the direction of minor axis.

Properties of 90° corner $R = 1{\cdot}6 + 0{\cdot}8 = 2{\cdot}4$ mm

$L = 1{\cdot}57 \times 2{\cdot}4 = 3{\cdot}77$ mm

$c = 0{\cdot}637 \times 2{\cdot}4 = 1{\cdot}53$ mm

The moment of inertia of the corner about its own axis is neglected.

I_{xx} of the full section

Web $= \dfrac{14{\cdot}36^3}{12} = 246{\cdot}7\ \text{cm}^3$

Straight portion of the lips $= \dfrac{14{\cdot}36^3}{12} - \dfrac{12^3}{12} = 102{\cdot}7\ \text{cm}^3$

Four corners $= 4 \times 0{\cdot}377 \times (7{\cdot}31)^2 = 80{\cdot}6\ \text{cm}^3$

Flanges $= 2 \times 5{\cdot}36 \times (7{\cdot}42)^2 = 590{\cdot}2\ \text{cm}^3$

$\therefore$ Linear $I_{xx} = 1\,020{\cdot}2\ \text{cm}^3$

(*Continued*)

DESIGN EXAMPLE NO. 6 WALL STUD BRACED BY WALL SHEATHING-AXIAL COMPRESSION MEMBER — *Contd*

Actual I_{xx} = 1 020·2 × 0·16 = 163·2 cm^4

I_{yy} of full section

Element	*Length, L* cm	*x* cm	*Lx* cm^2	*Lx*^{2} cm^3
Web	14·36	0·08	1·148 8	0·09
Straight portion of lips	2 × 1·18 = 2·36	5·92	12·95	82·60
Near corner	2 × 0·377 = 0·754	0 167	0·126	0·02
Far corner	= 0·754	5·833	4·39	25·60
Flanges	2 × 5·36 = 10 72	3·0	32·16	96·48
	28·948		51·774 8	204·79

$$X_{cg} = \frac{51{\cdot}774\,8}{28{\cdot}948} = 1{\cdot}79 \text{ cm}$$

Moment of inertia of the flange about their centres of gravity

$$= \frac{2 \times 5{\cdot}36^3}{12} = 25{\cdot}7 \text{ cm}^3$$

Linear I_{yy} = 204·79 + 25·7 − 28·948 (1·79)^2

= 230·49 − 92·75 = 137·74 cm^3

Actual I_{yy} or I_2 = 137·74 × 0·16 = 22·04 cm^4

Full sectional properties

$$A = L \times t = 28{\cdot}95 \times 0{\cdot}16 = 4{\cdot}635 \text{ cm}^2$$

$$r_1 = r_{xx} = \sqrt{\frac{163{\cdot}2}{4{\cdot}635}} = 5{\cdot}94 \text{ cm}$$

$$r_2 = r_{yy} = \sqrt{\frac{22{\cdot}04}{4{\cdot}635}} = 2{\cdot}18 \text{ cm}$$

Computations of Q, w/t of flange = 53·6/1·6 = 33·5

w/t of web = 143·6/1·6 = 89·75

b/t for flange (stress 1 250 kgf/cm^2) from Fig. 25B in Section 2 — full section effective.

b/t for web (stress 1 250 kgf/cm^2) from Fig. 25B in Section 2 = 51·25

b = 51·25 × 0·16 = 8·2 cm

Reduction = 14·36 − 8·2 = 6·16 cm

(*Continued*)

DESIGN EXAMPLE NO. 6 WALL STUD BRACED BY WALL SHEATHING-AXIAL COMPRESSION MEMBER — *Contd*

Total effective length $= 28{\cdot}948 - 6{\cdot}16 = 22{\cdot}788$ cm

$$Q = \frac{A_{eff}}{A} = \frac{L_{eff}}{L} = \frac{22{\cdot}788}{28{\cdot}948} = 0{\cdot}787\,2$$

$$\frac{L}{r_{xx}} = \frac{480}{5{\cdot}94} = 80{\cdot}8$$

Allowable stress from Fig. 33 in Section 2 $= 740$ kgf/cm^2

$$\therefore\ P_s = 740 \times 4{\cdot}635 = 3\,430 \text{ kgf}$$

Bracing requirement according to 8.1 of IS : 801-1975.

$$a_{max} = \frac{8\ EI_2 K_w}{A^2\ F_y{}^2}$$

$$= \frac{8 \times 2\,074\,000 \times 22{\cdot}05 \times 60}{(\,4{\cdot}635\,)^2 \times (\,2\,100\,)^2} = 231{\cdot}7 \text{ cm}$$

$$a_{max} = \frac{L.r_2}{2.r_1} = \frac{450 \times 2{\cdot}18}{2 \times 5{\cdot}94} = 82{\cdot}5 \text{ cm}$$

Provide a spacing of 82 cm for attachments and an end spacing of 75 mm at each end of the stud.

Force in the attachment $P_{Min} = \dfrac{K_w P_s \dfrac{L}{240}}{2 \times \sqrt{EI_2 \dfrac{K_w}{a}} - P_s}$

$$= \frac{60 \times 3\,430 \times \dfrac{450}{240}}{2 \times \sqrt{2\,074\,000 \times 22{\cdot}04 \times \dfrac{60}{82}} - 3\,430}$$

$$= \frac{385\,875}{(\,11\,569{\cdot}3 - 3\,430\,)} = \frac{385\,875}{8\,139} = 47{\cdot}4 \text{ kgf}$$

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS

Data

Structure: Welded ' W ' type roof truss

Span: 16 metres

Rise: Inclination of top chord is 15° to horizontal

Spacing of trusses: 2·5 metres centre-to-centre

Materials: Truss and bearing : Steel with a minimum yield stress = 23·2 kgf/mm²

Basic design stress = 1 250 kgf/cm²

Roofing: Slate, asbestos shingles, or built up roofing

Deck: Hollow light weight precast concrete plank

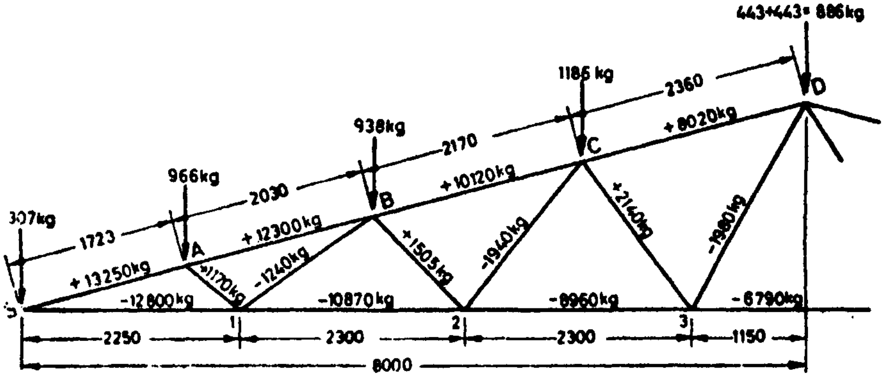
Loads:

Dead Loads:

Light weight concrete plank =	78 kgf/m²
Roofing =	19 kgf/m²
Assumed weight of truss and bracing =	25 kgf/m²
Total direct load	122 kgf/m²
Live load according to Table II of IS : 875-1964 = 75 − 5 =	70 kgf/m²
Total load =	192 kgf/m²

Total load per truss = 16 × 2·5 × 192
= 7 680 kgf

$$\text{Load per metre run} = \frac{7\ 680}{2 \times 8{\cdot}293} = 463 \text{ kgf}$$



All dimensions in millimetres.

FIG. 46

(Continued)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*

Local bending moments in the top chord

a) *Distribution factors* (*see* Fig. below)

Joint A

$$D_{AO} = \frac{2{\cdot}03}{0{\cdot}75 \times 1{\cdot}733 + 2{\cdot}03} = \frac{2{\cdot}03}{3{\cdot}33} = 0{\cdot}61$$

$$D_{AB} = 0{\cdot}39$$

Joint B

$$D_{BA} = \frac{2{\cdot}17}{2{\cdot}03 + 2{\cdot}17} = 0{\cdot}52$$

$$D_{BC} = 0{\cdot}48$$

Joint C

$$D_{CB} = \frac{0{\cdot}75 \times 2{\cdot}36}{0{\cdot}75 \times 2{\cdot}36 + 2{\cdot}17} = \frac{1{\cdot}77}{1{\cdot}77 + 2{\cdot}18} = \frac{1{\cdot}77}{3{\cdot}95} = 0{\cdot}45$$

$$D_{CD} = 0{\cdot}55$$

b) *Fixed end moments*

$$F_{OA} = -F_{AO} = \frac{463 \times 1{\cdot}733^2}{12} = 116 \text{ kgf.m}$$

$$F_{AB} = -F_{BA} = \frac{463 \times 1{\cdot}733^2}{12} = 159 \text{ kgf.m}$$

$$F_{BC} = -F_{CB} = \frac{463 \times 2{\cdot}17^2}{12} = 182 \text{ kgf.m}$$

$$F_{CD} = -F_{DC} = \frac{463 \times 2{\cdot}36^2}{12} = 215 \text{ kgf.m}$$

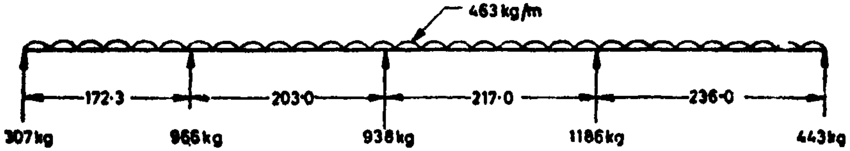


Fig. 47

(*Continued*)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*

Moment Distribution Table

Distribution Factor	Joints							
	S	A		B		C		D
	0	0·61	0·39	0·52	0·48	0·45	0·55	0
Fixed End Moment:	+ 116	– 116	+ 159	– 159	+ 182	– 182	+ 215	– 215
	– 116	—	—	—	—	—	—	+ 215
	—	– 58	—	—	—	—	+ 108	—
	—	+ 35	+ 23	—	—	– 49	– 59	—
	—	– 139	+ 182	– 159	+ 182	– 231	+ 264	—
	—	– 26	– 17	– 12	– 11	– 14	– 19	—
	—	—	– 6	– 8·5	– 7·0	– 5·5	—	—
	—	+ 3·7	+ 2·3	+ 8·0	+ 7·5	+ 2·5	+ 3	—
	—	—	+ 4·0	+ 1·2	+ 1·3	+ 3·8	—	—
	—	– 2·4	– 1·6	– 1·3	– 1·2	– 1·7	– 2·1	—
	—	—	– 0·7	– 0·8	– 0·9	– 0·6	—	—
	—	+ 0·4	+ 0·3	+ 1·0	+ 0·7	+ 0·27	+ 0·33	—
	—	—	+ 0·5	+ 0·15	+ 0·14	+ 0·4	—	—
	—	– 0·3	– 0·2	– 0·15	– 0·14	– 0·18	– 0·22	—
Support moments: in kgf/m		– 163·6	+ 163·6	– 171·4	+ 171·4	– 246·01	+ 246·01	—

The support reactions are calculated as shown, treating the rafter *S—A—B—C—D* as a continuous beam these reactions have been taken as the loads at the nodal points of the truss.

(*Continued*)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*

Table of lengths and forces — Forces in the various members as determined by the method of sections are tabulated below:

Member	*Length* cm	*Tension* kgf	*Compression* kgf
Top Chord			
S-A	172·3	—	13 250
A-B	203·0	—	12 300
B-C	217·0	—	10 210
C-D	236·0	—	8 020
Bottom Chord			
0-1	225·0	12 800	—
1-2	230·0	10 870	—
2-3	230·0	8 960	—
3-4	230·0	6 790	—
Diagonals			
A-1	73·0	—	1 170
B-1	168·0	1 240	—
B-2	132·7	—	1 500
C-2	194·0	1 640	—
C-3	190·6	—	2 140
D-3	245·4	1 980	—

Design of Top Chord

Assume a section as shown

$$w_1/t = \frac{108 - 16}{4} = 92/4 = 23$$

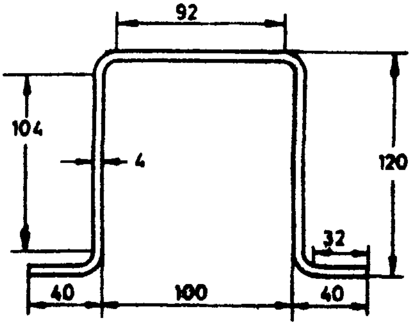
$$w_2 = 120 - 16 = 104$$

$$\frac{w_2}{t} = \frac{104}{4} = 26$$

$$(w/t)_{\text{lim}} = 1\,435/\sqrt{1\,250} = 40{\cdot}58$$

Hence all the elements are fully effective.

$\therefore\ Q = 1$



All dimensions in millimetres.

Fig. 48

Linear properties $L = 2 \times 3{\cdot}2 + 9{\cdot}2 + 2 + 10{\cdot}4 \times 4 \times 0{\cdot}942$

$= 40{\cdot}168$ cm

$A = 40{\cdot}168 \times 0{\cdot}4 = 16{\cdot}0672 = 16{\cdot}07\ \text{cm}^2$

(*Continued*)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*

Weight per metre = 16·07 × 0·785 = 12·6 kg

To find C_y ; $L.y$ = 9·2 × 0·2 + 2 × 10·4 × 6·0 + 2 × 3·2 × 11·8 + 2 × 0·942 × 0·545 + 2 × 0·942 × 11·455 = 224·766 cm²

$$y = \frac{224{\cdot}766}{40{\cdot}168} = 5{\cdot}6 \text{ cm}$$

$$\text{Linear } I_{xx} = 9{\cdot}2 \times (5{\cdot}4)^2 + 2 \times \frac{10{\cdot}4^3}{12} + 2 \times 10{\cdot}4 \times (0{\cdot}4)^2 + 2 \times 3{\cdot}2 \times (6{\cdot}2)^2 + 2 \times 0{\cdot}942 \times (5{\cdot}055)^2 + 2 \times 0{\cdot}942 \times (5{\cdot}855)^2 \text{ cm}^3$$

$$= 268{\cdot}1 + 187{\cdot}5 + 3{\cdot}33 + 246{\cdot}0 + 48{\cdot}2 + 68{\cdot}15 \text{ cm}^3$$

$$= 821{\cdot}28 \text{ cm}^3 \text{ say } 821 \text{ cm}^3$$

$$\text{Actual } I_{xx} = 821 \times 0{\cdot}4$$

$$= 328 \text{ cm}^4$$

$$\text{Linear } I_{yy} = \frac{9{\cdot}2^3}{12} + 2 \times \frac{3{\cdot}2^3}{12} + 2 \times 3{\cdot}2 \times (7{\cdot}4)^2 + 2 \times 10{\cdot}4 \times (5{\cdot}2)^2 + 2 \times 0{\cdot}942 \times (4{\cdot}855)^2 + 2 \times 0{\cdot}942 \times (5{\cdot}545)^2 \text{ cm}^3$$

$$= 64{\cdot}9 + 5{\cdot}46 + 350{\cdot}1 + 563{\cdot}0 + 44{\cdot}5 + 57{\cdot}9 \text{ cm}^3$$

$$= 1\,085{\cdot}86 \text{ cm}^3$$

$$\text{Actual } I_{yy} = 1\,085{\cdot}86 \times 0{\cdot}4 = 434{\cdot}344 \text{ cm}^4$$

$$r_{xx} = \sqrt{\frac{328}{16{\cdot}07}}$$

$$= 4{\cdot}525 \text{ cm}$$

Span OA

$$\frac{L}{r} = \frac{172{\cdot}3}{4{\cdot}525} = 38{\cdot}1 \; ; \; Q = 1$$

Allowable compressive stress F_{a1} from Fig. 41 in Section 2 is 1 145 kgf/cm²

$$\text{Actual compressive stress } f_a = \frac{13\,250}{16{\cdot}07} = 825 \text{ kgf/cm}^2$$

$$\text{Actual bending stress } f_b = \frac{163{\cdot}6 \times 100 \times 5{\cdot}6}{328} = 279 \text{ kgf/cm}^2$$

F_b = allowable bending stress = 1 250 kgf/cm²

Checking for the combined axial and bending stress condition (according to 6.7.2 of IS : 801-1975)

$$\frac{f_a}{F_{a1}} + \frac{f_b}{F_b\left[1 - \frac{f_a}{F'_e}\right]} < 1 \quad (\text{taking } C_m = 1)$$

where

$$F'_e = \frac{12\pi^2 E}{23\,(L_b/r_b)^2} = \frac{12 \times 9{\cdot}87 \times 2\,074\,000}{23 \times (38{\cdot}1)^2} = 7\,357{\cdot}48$$

(Continued)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*

Hence

$$\frac{f_a}{F_{al}} + \frac{f_b}{F_b\left[1-\frac{f_a}{F'_e}\right]} = \frac{825}{1\,145} + \frac{279}{1\,250 \times 0\cdot89} = 0\cdot720\,5 + 0\cdot250\,8 = 0\cdot971\,3 < 1$$

The section is O. K.

Span AB An effective length factor of 0·85 can be assumed as the member is continuous.

$$\frac{L}{r} = \frac{0\cdot85 \times 203}{4\cdot525} = 38\cdot25$$

F_{al} from Fig. 41 in Section 2 = 1 145 kgf/cm²

$$\text{Actual } f_a = \frac{12\,300}{16\cdot07} = 765 \text{ kgf/cm}^2 \qquad \frac{f_a}{F'_e} = \frac{765}{7\,357} = 0\cdot10$$

$$F_b = \frac{171\cdot4 \times 100 \times 6\cdot4}{328\cdot512} = 334\cdot0 \text{ kgf/cm}^2$$

$$\frac{f_a}{F_{al}} + \frac{f_b}{(1 - f_a/F'_e)F_b} = \frac{765}{1\,145} + \frac{334}{0\cdot90 \times 1\,250}$$

$$= 0\cdot668\,1 + 0\cdot296\,9 = 0\cdot965\,0 < 1$$

The section is O. K.

Bottom Chord

To facilitate easy fabrication, the bottom chord will be made continuous from point *O* to point 3, and a field splice will be provided in the centre of span 3-4.

Maximum force in the bottom chord = 12 800 kgf

$$\text{Area required} = \frac{12\,800}{1\,250} = 10\cdot25 \text{ cm}^2$$

A channel will be chosen that has a total width of 100 mm so that it will fit into the top chord hat section to ease the connections at the end supports.

$$\begin{aligned}\text{Total length} &= 84 + 164 + 2 \times 9\cdot42 \\ &= 248 + 18\cdot84 \\ &= 266\cdot84 \text{ mm} \\ &= 26\cdot7 \text{ cm}\end{aligned}$$

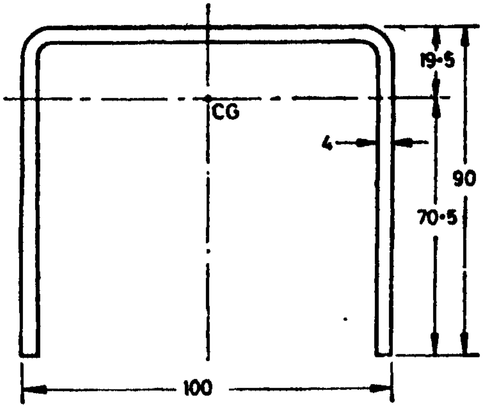
Area = 26·7 × 0·4 = 10·68 cm² > 10·25 cm² actually required.

The centre of gravity of the section is at

$$\begin{aligned}&(8\cdot4 \times 0\cdot2 + 8\cdot2 \times 4\cdot9 + 2 \times 0\cdot942 \times 5\cdot45)\,\frac{1}{26\cdot7} \\ &= (1\cdot68 + 40\cdot15 + 10\cdot28)\,\frac{1}{26\cdot7} \\ &= 52\cdot11 \times \frac{1}{26\cdot7} = 1\cdot95 \text{ cm from top.}\end{aligned}$$

(*Continued*)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*



All dimensions in millimetres.

FIG. 49

Linear $I_{xx} = 8{\cdot}4 \times (1{\cdot}75)^2 + 2 \times 0{\cdot}942 \times (1{\cdot}4)^2 + \frac{8{\cdot}2^3}{12} + 8{\cdot}2 \times (2{\cdot}95)^2$

$= 24{\cdot}25 + 3{\cdot}69 + 46{\cdot}00 + 71{\cdot}3$

$= 145{\cdot}24 \text{ cm}^3$

Actual $I_{xx} = 145{\cdot}24 \times 0{\cdot}4$

$= 58{\cdot}096 \text{ cm}^4$

Linear $I_{yy} = 8{\cdot}4^3/12 + 2 \times 0{\cdot}942 \times (4{\cdot}455)^2 + 2 \times 8\,2 \times (4{\cdot}8)^2$

$= 49{\cdot}4 + 37{\cdot}5 + 377{\cdot}8$

$= 464{\cdot}7 \text{ cm}^3$

Actual $I_{yy} = 464{\cdot}7 \times 0{\cdot}4$

$= 185{\cdot}88 \text{ cm}^4$

$$r_{xx} = \sqrt{\frac{58{\cdot}096}{26{\cdot}7 \times 0{\cdot}4}} = 2{\cdot}335 \text{ cm} \quad \frac{L}{r_{xx}} = \frac{225}{2{\cdot}335} = 196{\cdot}5$$

< 350 hence O.K.

Splicing of bottom chords

Bolted connection will be adopted to ease erection.

Force in point 3 – 4 = 6 790 kgf

(*Continued*)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*

50 percent of effective
strength of member $= 0{\cdot}5 \times 10{\cdot}25 \times 1\,250$
$= 6\,400$ kgf

Therefore, 6 790 kgf governs.

Minimum bearing stress of bolt $= 3{\cdot}5\, f_b$
$= 3{\cdot}5 \times 1\,250 = 4\,375$ kgf/cm²

Minimum shear stress of bolt $= \dfrac{3\,400}{4} = 850$ kgf/cm²

If three bolts are provided one in each flange and the third in the web; strength of each bolt $= \dfrac{6\,790}{3} = 2\,263$ kgf

If 16 mm diameter bolts are adopted:
Root area = 1·57 cm²

Strength of each bolt
in double shear $= 2 \times 1{\cdot}57 \times 850 = 2\,670$ kgf

Therefore use three 16 mm dia bolts

Allowable stress on the net section (7.5.2 of IS : 801-1975)

$$= (1{\cdot}0 - 0{\cdot}9r + 3rd/s)\, 0{\cdot}6\, F_y$$

$$= \left[1{\cdot}0 - 0{\cdot}9 + \frac{3 \times 1 \times 1{\cdot}6}{(10 + 9)/2} \right] 1\,250 = 750 \text{ kgf/cm}^2;\ r \text{ being } = 1$$

Net section of the member = gross area – area of holes
$= (10{\cdot}25 - 3 \times 1{\cdot}6 \times 0{\cdot}4) = 10{\cdot}25 - 1 - 92$
$= 8{\cdot}33$ cm²

Actual stress on the net section $= \dfrac{6\,790}{8{\cdot}33} = 815$ kgf/cm²
> 750 kgf/cm²

Therefore, reinforcing plate is necessary.
Try a plate 65 × 4 mm
Gross area $= 10{\cdot}25 + 6{\cdot}5 \times 0{\cdot}4 = 12{\cdot}85$ cm²
Area of holes $= 4 \times 1{\cdot}6 \times 0{\cdot}4 = 2{\cdot}56$ cm²
Net area $= 12{\cdot}85 - 2{\cdot}56 = 10{\cdot}29$ cm²
Stress on the net section $= 6\,790/10{\cdot}29 = 660$ kgf/cm² $<$ 750 kgf/cm² allowable.

Allowable stress on the net section of splice plate $= \left(1{\cdot}0 - 0{\cdot}9 + \dfrac{3 \times 1{\cdot}6}{6{\cdot}5}\right) 1\,250$;
r being 1·0
$= 1\,030$ kgf/cm²

Providing 65 × 4 mm plates for web and flanges:
Gross area of splice plate $= 6 \times 6{\cdot}5 \times 0{\cdot}4 = 15{\cdot}6$ cm²
Deduction for holes $= 6 \times 1{\cdot}75 \times 0{\cdot}4 = 4{\cdot}2$ cm²

(*Continued*)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*

Net area $= 15·6 - 4·2 = 11·4$ cm^2

Stress on net section $= \dfrac{6\ 790}{11·4} = 595$ kgf/cm^2

$< 1\ 050$ kgf/cm^2.........O. K.

Check for bearings

Bearing stress $= \dfrac{6\ 790}{4 \times 0·4 \times 1·6} = 265$ kgf/cm^2 ....allowable

Edge distance (*see* 7.5.1 of IS : 801-1975)

$= \dfrac{2\ 263}{0·6 \times 2\ 100 \times 0·4} = 4·49$ cm

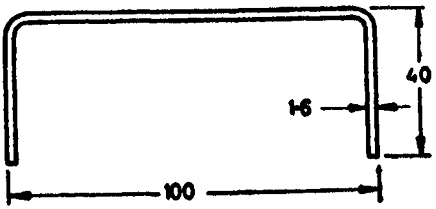
Provide 5 cm edge distance

Tension diagonals — Members B-1, C-2 and D-3

Maximum force = 1 980 kg. Maximum length = 245·4 cm

For ease of fabrication all tension diagonals will be of the same section. The section will have an outside width of 100 mm so as to fit into the top chord to enable easy welding.

Area required $= \dfrac{1\ 980}{1\ 250} = 1·585$ cm^2



All dimensions in millimetres.

FIG. 50

Channel 100 × 40 × 1·6 (conforming to Table III of IS : 811-1965)

Area $= 2·763$ cm^2

$I_{xx} = 41·429$ cm^4.

$r_{xx} = 3·87$ cm

$\dfrac{L}{r_{xx}} = \dfrac{245·4}{3·87} = 85·0 < 180$......O.K.

Compression diagonals

Members A-1 and B-2

Maximum force 1 505 kg, length = 132·7 cm

A channel 100 × 40 × 2·0 according to Table III of IS : 811-1965 is proposed.

$A = 3·434$ cm^2
$I_{xx} = 51·032$ cm^4
$I_{yy} = 5·212$ cm^4
$r_{xx} = 3·85$ cm
$r_{yy} = 1·23$ cm
$Q = 0·813$
$L/r = 132·7/1·23 = 108$ for r value of $Q = 0·813$

From Fig. 37, Allowable stress $F_a = 670$ kgf/cm^2

$f_a = 1·505/3·434 = 439$ kgf/cm$^2 < 670$ kgf/cm^2 ...too uneconomical

(*Continued*)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*

Try 100 × 40 × 1·6 channel

I_{xx} = 41·429 cm^4
I_{yy} = 4·233 cm^4
r_{xx} = 3·87 cm
r_{yy} = 1·24 cm
A = 2·76 cm^2
Q = 0·66
L/r = 132·7/1·24 = 107

∴ Allowable stress from Fig. 37 is 580 kgf/cm^2

f_a = 1 505/2·76 = 595 kgf/cm^2 > 580 kgf/cm^2.........O. K.

Member D-3

Try a lipped channel 100 × 50 × 1·6 mm according to Table V of IS : 811-1965.

A = 3·446 cm^2
I_{xx} = 55·045 cm^4
I_{yy} = 11·962 cm^4
r_{xx} = 4·00 cm
r_{yy} = 1·86 cm
Q = 0·901

$$\frac{L}{r_{yy}} = \frac{190 \cdot 6}{1 \cdot 86} = 102 \cdot 3$$

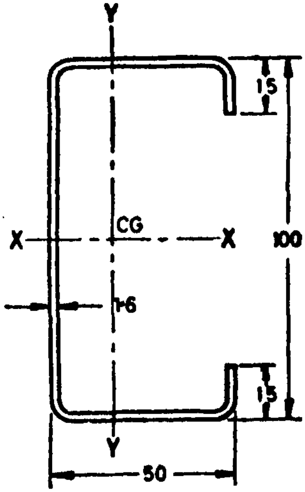
F_a = from Fig. 37 is 750 kgf/cm^2

$$\text{Actual compression stress} = \frac{2\,140}{3 \cdot 446}$$

$$= 620 \text{ kgf/cm}^2$$

$$< 750 \text{ kgf/cm}^2$$

Therefore, provide a lipped channel
100 × 50 × 1·6 mm



All dimensions in millimetres.

FIG. 51

Comparison of conventional and light-gauge designs

The roof truss of Design Example No. 7 is designed below by conventional method using hot rolled sections.

Top chord

Members	*Axial Comparison* kgf	*Design Moment* cm.kgf	*Length* cm
OA	13 250		172·3
		M_A 163·6 × 10^3	
AB	12 300		203·0
		M_B 171·4 × 10^3	
BC	10 120		217·0
		M_C 246·01 × 10^3	
CD	8 020		236·0

(*Continued*)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*

The design is done for panel *BC*, and the section is checked for adequacy for panel *AB*.

Panel BC

Effective length $l = 213 \times 0{\cdot}85 = 184{\cdot}5$ cm

$M = 246{\cdot}01 \times 10^2$ cm.kgf,

$P = 10\ 120$ kgf

Try ISMB 150

$t_f = 7{\cdot}6$ mm

$A = 19{\cdot}0$ cm^2

$I^1_{yy} = I_{yy} = 52{\cdot}6$ cm^4

$r_{yy} = 1{\cdot}66$ cm

$Z_x = 96{\cdot}9$ cm^3

$h = 150 - 7{\cdot}6 = 142{\cdot}6$ cm

$K = 2{\cdot}8$ cm^4

$$\frac{I^1_y . h}{Z_x\, l^2} = \frac{52{\cdot}6 \times 14{\cdot}26}{96{\cdot}9 \times (\ 184{\cdot}5\ )^2}$$

$$= 0{\cdot}000\ 227\ 5$$

$$\frac{K\, l^2}{I^1 . h^2} = \frac{2{\cdot}8 \times (\ 184{\cdot}5\ )^2}{52{\cdot}6 \times (\ 14{\cdot}26\ )^2}$$

$$= 8{\cdot}92$$

From Table V of IS : 800-1962,

$C_s = 3\ 584$ kgf/cm^2

According to 10.2.2.1 of IS : 800-1962, this has to be increased by 20 percent for rolled sections.

Therefore, $C_s = 3\ 584 + 717 = 4\ 301$ kgf/cm^2

$$l/r_{yy} = \frac{184{\cdot}5}{1{\cdot}66} = 111$$

$F_b = 1\ 565$ kgf/cm^2 from Table IV of IS : 800-1962

$$\frac{L}{r_{yy}} = \frac{217}{1{\cdot}66} = 131$$

Therefore, $F_a = 590$ kgf/cm^2

$$f_a = \frac{10\ 120}{19} = 532 \text{ kgf/cm}^2$$

$$f_b = \frac{24\ 601}{96{\cdot}9} = 254 \text{ kgf/cm}^2$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{532}{590} + \frac{254}{1\ 565}$$

$$= 0{\cdot}902 + 0{\cdot}162 = > 1$$

(*Continued*)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*

Panel CD

$M = 246{\cdot}01 \times 10^2$ cm.kgf; $P = 8\,020$ kgf
$L = 236$ cm, effective length of compression flange $= 236 \times 0{\cdot}85 = 200$ cm
From Table xxvii of IS : 800-1962 permissible bending stress $F_b = 1\,557$ kgf/cm²

$$L/r_{yy} = 236/1{\cdot}66 = 142$$
$$F_a = 520 \text{ kgf/cm}^2$$
$$f_b = \frac{246{\cdot}01 \times 10^2}{96{\cdot}9} = 254 \text{ kgf/cm}^2$$
$$a = \frac{8\,020}{19} = 422 \text{ kgf/cm}^2$$
$$\frac{f_a}{F} + \frac{f_b}{F_b} = \frac{422}{520} + \frac{254}{1\,557}$$
$$= 0{\cdot}813 + 0{\cdot}163$$
$$= 0{\cdot}976 < 1.$$

Hence ISMB 150 may be used as the top chord.

Bottom chord

Maximum tensile force = 12 800 kgf

Approximate area required $= \frac{12\,800}{1\,500} = 8{\cdot}53$ cm²

Use ISLC 100 × 7·9 kg/m
Area = 10·02 cm²
$\frac{L}{r} = \frac{225}{1{\cdot}57} = 143 < 350$ ........O.K.

Tension diagonals

Maximum tensile force = 1 980 kgf

Minimum radius of gyration required $= \frac{245{\cdot}4}{180} = 1{\cdot}36$ cm

Use ISA 70 × 70 × 5 mm

Area = 6·77

$r_{min} = 1{\cdot}35$ cm

Net area

$a = 7{\cdot}0 \times 0{\cdot}5 = 3{\cdot}5$ cm²
$b = 6{\cdot}5 \times 0{\cdot}5 = 3{\cdot}25$ cm²
The net area of the angle section is calculated according to **20.3.1** of IS : 800-1962.

Net area of the section $= a + kb = 3{\cdot}5 + \frac{1}{1 + 0{\cdot}35 \times 3{\cdot}25/3{\cdot}5} \times 3{\cdot}25$
$= 6{\cdot}95$ cm²

Use ISA 70 × 70 × 5 mm

(Continued)

DESIGN EXAMPLE NO. 7 WELDED COLD-FORMED LIGHT-GAUGE STEEL ROOF TRUSS — *Contd*

Compression diagonals

Maximum compression force = 2·14 Tensile

Maximum length = 190·6 cm

$$\text{Minimum radius of gyration} = \frac{190{\cdot}6 \times 0{\cdot}85}{180} = 0{\cdot}9 \text{ cm}$$

Try ISA 60 × 60 × 5 mm

$r_{min} = 1{\cdot}16;$

Area $= 5{\cdot}75 \text{ cm}^2$

$$\frac{L}{r} = \frac{190{\cdot}06 \times 0{\cdot}85}{1{\cdot}16} = 140$$

$F_a = 531 \text{ kgf/cm}^2$

$$f_a = \frac{2\,140}{5{\cdot}75} = 372 \text{ kgf/cm}^2$$

$< 531 \text{ kgf/cm}^2$ .... too uneconomical.

Try ISA 55 × 55 × 5 mm

$r_{min} = 1{\cdot}06$ cm

Area $= 5{\cdot}27 \text{ cm}^2$

$$\frac{L}{r_{min}} = \frac{0{\cdot}85 \times 190{\cdot}6}{1{\cdot}06} = 152{\cdot}5$$

$F_a = 462 \text{ kgf/cm}^2$

$$f_a = \frac{2\,140}{5{\cdot}27} = 406 \text{ kgf/cm}^2 < 462 \text{ kgf/cm}^2$$ ...... O. K.

Comparison of weights

Member	*Hot Rolled Sections* kg		*Light-Gauge Cold Rolled Sections* kg	
Top chord	2 × 8·283 × 14·9	= 246·8	2 × 8·283 × 12·6	= 208·0
Bottom chord	16 × 7·9	= 127·0	16 × 8·38	= 134
Tension diagonals	12·148 × 5·3	= 64·5	12·148 × 2·18	= 27
Compression	7·24 × 4·1	= 29·7	7·24 × 2·7	= 19·5
Diagonals	3·82 × 4·1	= 15·7	3·82 × 2·705	= 10·5
		483·7		399·1
Add 10 percent for gusset plates		51·1		—
		534·8		399·1

$$\text{Saving in steel} = \frac{535 - 399}{564} \times 100 = \frac{136}{564} \times 100 = 24{\cdot}2 \text{ percent}$$

NOTE — The comparison of weight is only of academic interest, as the cost of 1 tonne of cold-formed structures is about twice that of hot-rolled sections.

APPENDIX A

(*Clause* 0.1)

COMPOSITION OF STRUCTURAL ENGINEERING SECTIONAL COMMITTEE, SMBDC 7

The ISI Structural Engineering Sectional Committee, SMBDC 7, which is responsible for processing this Handbook, consists of the following:

Chairman	*Representing*
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Members	
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Dr Prem Krishna (*Alternate*)	
Shri A. K. Banerjee	Metallurgical and Engineering Consultants (India) Ltd, Ranchi
Shri S. Sankaran (*Alternate*)	
Shri B. G. Barduan	Braithwaite & Co Ltd, Calcutta
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Shri S. N. Basu	Inspection Wing, Directorate General of Supplies and Disposals, New Delhi
Shri D. B. Jain (*Alternate*)	
Shri P. C. Bhasin	Ministry of Shipping and Transport (Department of Transport) (Roads Wing)
Shri V. S. Bhide	Central Water Commission, New Delhi
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Dr P. N. Chatterjee	Government of West Bengal
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Shri A. B. Ribeiro (*Alternate*)	
Shri N. V. Raman	Structural Engineering Research Centre (CSIR), Madras
Dr T. V. S. R. Appa Rao (*Alternate*)	

(*Continued on page* 120)

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Members	*Representing*
Prof H. C. Parameswar	Engineer-in-Chief's Branch, Ministry of Defence
Shri C. S. S. Rao (*Alternate*)	
Shri Dilip Paul	Industrial Fasteners Association of India, Calcutta
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Dr D. N. Trikha	University of Roorkee, Roorkee
Shri L. D. Wadhwa	Engineers India Ltd, New Delhi
Shri B. B. Nag (*Alternate*)	
Shri C. R. Rama Rao, Director (Struc & Met)	Director General, ISI (*Ex-officio Member*)

Secretary

Shri S. S. Sethi
Assistant Director (Struc & Met), ISI

Panel for the Revision of Handbook on Cold-Formed Light Gauge Steel Structures, SMBDC 7 : P 31

Dr C. Ganapathy Chettiar	Indian Institute of Technology, Madras

APPENDIX B

(*Clause* 0.3)

LIST OF IMPORTANT STANDARDS AND CODES OF PRACTICES PUBLISHED BY THE INDIAN STANDARDS INSTITUTION IN THE FIELD OF STEEL PRODUCTION, DESIGN AND USE

I *Materials*

a) *Structural Steel*

IS:	
226-1975	Structural steel (standard quality) (*fifth revision*)
961-1975	Structural steel (high tensile) (*second revision*)
1161-1968	Steel tubes for structural purposes (*second revision*)
1977-1975	Structural steel (ordinary quality) (*first revision*)
2062-1969	Structural steel (fusion welding quality) (*first revision*)

b) *Structural Shapes (Steel)*

IS:

808-1964	Rolled steel beam, channel and angle sections (*revised*)
808 (Part I)-1973	Dimensions of hot rolled steel sections — MB series (beams)
808 (Part II)-1976	Dimensions of hot rolled steel columns — SC series (*second revision*)
808 (Part V)-1976	Dimensions of hot rolled steel sections: Part V Equal leg angles (*second revision*)
808 (Part VI)-1976	Dimensions of hot rolled steel sections: Part VI Unequal leg angles (*second revision*)
811-1965	Cold-formed light gauge structural steel sections (*revised*)
1161-1968	Steel tubes for structural purposes (*second revision*)
1730 (Part I)-1975	Dimensions for steel plate, sheet, strip for structural and general engineering purposes: Part I Plate (*first revision*)
1730 (Part II)-1975	Dimensions for steel plate, sheet, strip for structural and general engineering purposes: Part II Sheet (*first revision*)
1730 (Part III)-1975	Dimensions for steel plate, sheet, strip for structural and general engineering purposes: Part III Strip (*first revision*)
1731-1971	Dimensions for steel flats for structural and general engineering purposes (*first revision*)
3954-1966	Hot rolled steel channel sections for general engineering purposes

c) *Fasteners (Rivets/Bolts)*

IS:

730-1966	Fasteners for corrugated sheet roofing (*revised*)
1364-1967	Precision and semi-precision hexagon bolts, screws, nuts and lock nuts (dia range 6 to 39 mm) (*first revision*)
1367-1967	Technical supply conditions for threaded fastener (*first revision*)
1821-1967	Dimensions for clearance holes for metric bolts (*first revision*)
1862-1975	Studs (*first revision*)
1929-1961	Rivets for general purposes (below 12 mm to 48 mm diameter)
2016-1967	Plain washers (*first revision*)
2155-1962	Rivets for general purposes (below 12 mm diameter)
2389-1968	Precision hexagon bolts, screws, nuts and lock nuts (dia range 1·6 to 5 mm) (*first revision*)

IS:	
2585-1968	Black square bolts and nuts (diameter range 6 to 39 mm) and black square screws (diameter range 6 to 24 mm) (*first revision*)
2609-1972	Coach bolts (*first revision*)
2687-1975	Cap nuts (*first revision*)
2998-1965	Cold forged steel rivets for cold closing
3063-1972	Single coil rectangular section spring washers for bolts, nuts and screws (*first revision*)
3138-1966	Hexagonal bolts and nuts
3139-1966	Dimensions for screw threads for bolts and nuts
3640-1967	Hexagon fit bolts
3757-1972	High-tensile friction grip bolts (*first revision*)
4206-1967	Dimensions for nominal lengths and thread lengths for bolts screws and studs
5370-1969	Plain washers with outside diameter ≈ 3 × inside diameter
5371-1969	Multi-tooth lock washers
5372-1975	Taper washers for channels (ISMC) (*first revision*)
5374-1975	Taper washers for I-beams (ISMB) (*first revision*)
5554-1970	Lock washers with lug
6610-1972	Heavy washers for steel structures
6623-1972	High tensile friction grip nuts
6639-1972	Hexagon bolts for steel structures
6649-1972	High tensile friction grip washers

II *Design Code*

IS:	
800-1962	Code of practice for use of structural steel in general buildin construction (*revised*)
801-1975	Code of practice for use of cold-formed light gauge stee structural members in general building construction
802 (Part I)-1977	Code of practice for use of structural steel in overhead transmission line towers: Part I Loads and permissibl stresses (*first revision*)
802 (Part II)-1978	Code of practice for use of structural steel in overhead transmission line towers: Part II Fabrication, galvanizing, inspection and packing
802 (Part III)-1978	Code of practice for use of structural steel in overhead transmission line towers: Part III Testing
803-1976	Code of practice for design, fabrication and erection of vertical mild steel cylindrical welded oil storage tanks
804-1967	Rectangular pressed steel tanks (*first revision*)
805-1968	Code of practice for use of steel in gravity water tanks

IS:

806-1968	Code of practice for use of steel tubes in general building construction
807-1976	Code of practice for design, manufacture, erection and testing (structural portion) of cranes and hoists (*first revision*)
3177-1977	Code of practice for electric overhead travelling cranes and gentry cranes other than steel work cranes
4000-1967	Code of practice for assembly of structural joints using high tensile friction grip fasteners
4573-1968	Code of practice for design of mobile cranes (all types)
4594-1963	Code of practice for design of portal and semi-portal wharf cranes (electrical)
6409-1971	Code of practice for oxy-acetylene flame cleaning
6521 (Part I)-1972	Code of practice for design of tower cranes: Part I Static and rail mounted
6533-1972	Code of practice for design and construction of steel chimneys

III *Welding*

IS:

812-1957	Glossary of terms relating to welding and cutting of metals
813-1961	Scheme of symbols for welding (*amended*)
814 (Part I)-1974	Covered electrodes for metal arc welding of structural steel for welding products other than sheets (*fourth revision*)
814 (Part II)-1974	For welding sheets (*fourth revision*)
815-1974	Classification and coding of covered electrodes for metal arc welding of structural steels (*second revision*)
816-1969	Code of practice for use of metal arc welding for general construction in mild steel (*first revision*)
817-1966	Code of practice for training and testing of metal arc welders (*revised*)
818-1968	Code of practice for safety and health requirements in electric and gas welding and cutting operations (*first revision*)
819-1957	Code of practice for resistance spot welding for light assemblies in mild steel
822-1970	Code of procedure for inspection of welds
823-1964	Code of procedure for manual metal arc welding of mild steel
1024-1968	Code of practice for use of welding in bridges and structures subject to dynamic loading
1179-1967	Equipment for eye and face protection during welding (*first revision*)
1261-1959	Code of practice for seam welding in mild steel
1278-1972	Filler rods and wires for gas welding (*second revision*)

IS:

1393-1961	Code of practice for training and testing of oxy-acetylene welders
1395-1971	Molybdenum and chromium-molybdenum-vanadium low alloy steel electrodes for metal-arc welding (*second revision*)
2811-1964	Recommendations for manual tungsten inert-gas arc-welding of stainless steel
2812-1964	Recommendations for manual tungsten inert-gas arc-welding of aluminium and aluminium alloys
3016-1965	Code of practice for fire precautions in welding and cutting operations
3023-1965	Recommended practice for building-up metal spraying
3600 (Part I)-1973	Code of procedure for testing of fusion welded joints and weld metal in steel — General test (*first revision*)
3613-1970	Acceptance tests for wire flux combinations for submerged arc welding (*first revision*)
4353-1967	Recommendations for sub-merged arc welding of mild steel and low alloy steels
4943-1968	Assessment of butt and fillet fusion welds in steel sheet, plate and pipe
4944-1968	Code of procedure for welding at low ambient temperatures
4972-1968	Resistance spot-welding electrodes
5206-1969	Corrosion-resisting chromium and chromium-nickel steel covered electrodes for manual metal arc welding
5462-1969	Colour code for identification of covered electrodes for metal arc welding
5922-1970	Qualifying test for welders engaged in aircraft welding
6560-1972	Molybdenum and chromium-molybdenum low alloy steel welding rods and base electrodes for gas shielded arc welding
7307 (Part I)-1974	Approval testing of welding procedures: Part I Fusion welding of steel
7310 (Part I)-1974	Approval testing of welders working to approval welding procedures: Part I Fusion welding of steel
7318 (Part I)-1974	Approval testing of welders when welding procedure approval is not required: Part I Fusion welding of steel
7318 (Part II)-1974	Approval test for welders when welding procedure approval is not required: Part II TIG or MIG welding of aluminium and its alloys

IV) *Handbooks*

SP:

6(1)-1964	Structural steel sections (*revised*)
6(2)-1962	Steel beams and plate girders
6(3)-1962	Steel columns and struts

SP:

6(4)-1969	Use of high strength friction grip bolts
6(6)-1972	Application of plastic theory in design of steel structures
6(7)-1972	Simple welded girders
12-1975	ISI Handbook for gas welders

V) *Miscellaneous*

IS:

696-1972	Code of practice for general engineering drawings (*second revision*)
962-1967	Code of practice for architectural and building drawings (*first revision*)
7205-1974	Safety code for erection of structural steelwork
7215-1974	Tolerances for fabrication of steel structures
8640-1977	Recommendations for dimensional parameters for industrial buildings